6.1 Seismic Design Responsibility and Policy

6.1.1 Responsibility of the Geotechnical Designer

The geotechnical designer is responsible for providing geotechnical/seismic input parameters to the structural engineers for their use in structural design of the transportation infrastructure (e.g., bridges, retaining walls, ferry terminals, etc.). Specific elements to be addressed by the geotechnical designer include the design ground motion parameters, site response, and geologic hazards. The geotechnical designer is also responsible for providing input for evaluation of soil-structure interaction (foundation response to seismic loading), earthquake induced earth pressures on retaining walls, and an assessment of the impacts of geologic hazards on the structures.

6.1.2 Geotechnical Seismic Design Policies

6.1.2.1 Seismic Performance Objectives

In general, the AASHTO Load and resistance factor Design (LRFD) Bridge Design Specifications shall be followed for structure classification of bridges. Critical, essential, and other structures are defined in AASHTO LRFD Bridge Design Specifications. In the current inventory, most structures are considered "other" with a few being "essential" or "critical". In keeping with the current seismic design approaches employed both nationally and internationally, geotechnical seismic design shall be consistent with the philosophy for structure design that loss of life and serious injury due to structure collapse are minimized, to the extent possible and economically feasible. The definition of structure collapse is provided in the WSDOT LRFD Bridge Design Manual (BDM). Bridges, regardless of their AASHTO classification, may suffer damage and may need to be replaced after a design seismic event, but they should be designed for non-collapse due to earthquake shaking and geologic hazards associated with a design seismic event.

In keeping with the no collapse philosophy, bridge approach embankments and fills through which cut-and-cover tunnels are constructed should be designed to remain stable during the design seismic event because of the potential to damage or initiate collapse of the structure should they fail. The aerial extent of approach embankment seismic design and mitigation (if necessary) should be such that the structure is protected against instability or loading conditions that could result in collapse. The typical distance of evaluation and mitigation is within 100 feet of the abutment or tunnel wall. Instability or other seismic hazards such as liquefaction, lateral spread, downdrag, and settlement may require mitigation near the abutment or tunnel wall to ensure that the structure is not compromised during a design seismic event. The geotechnical designer should evaluate the potential for differential settlement between mitigated and non-mitigated soils. Additional measures may be required to limit differential settlements to tolerable levels both for static and seismic conditions. The bridge interior pier foundations should also be designed to be adequately stable with regard to liquefaction, lateral flow, and other seismic effects to prevent bridge collapse.

For the case where an existing bridge is to be widened and liquefiable soil is present, the foundations for the widened portion of the bridge and bridge approaches should be designed to remain stable during the design seismic event such that bridge collapse does not occur. In addition, if the existing bridge foundation is not stable, to the extent practical, measures should be taken to prevent collapse of the existing bridge during the design seismic event. The foundations for the widening should be designed in a way that the seismic response of the bridge widening can be made compatible with the seismic response of the existing bridge as stabilized in terms of foundation deformation and stiffness. If it is not feasible to stabilize the existing bridge such that it will not collapse during the design seismic event, consideration should be given to replacing the existing bridge rather than widening it.

All retaining walls and abutment walls shall be evaluated for seismic stability internally and externally (i.e. sliding and overturning). The geotechnical designer shall evaluate the impacts of failure due to seismic loading for all walls. Walls directly supporting the traveled way, or walls that are directly adjacent to the traveled way and are 10 ft in height or more, should be designed to remain stable under seismic loading conditions and anticipated displacements associated with liquefaction. Mitigation to achieve overall stability may be required.

Walls in which the wall face is more than 10 ft from the traveled roadway, and walls that are less than 10 ft in height, are not required to meet overall stability under seismic loading and/or liquefaction effects. These walls are considered to have relatively low risk to the traveling public. These walls may deform, translate, or rotate during a seismic event and overall stability may be compromised. Considering the excessive cost required to stabilize these walls for liquefaction effects, it is generally considered uneconomical to stabilize these lower risk walls for liquefaction.

Cut slopes in soil and rock, fill slopes, and embankments should be evaluated for instability due to design seismic events and associated geologic hazards. However, instability associated with cuts and fills are not mitigated due to the high cost of applying such a design policy uniformly to all slopes statewide. However, slopes that could impact an adjacent structure if failure due to seismic loading occurs should be stabilized.

Note that the policy to stabilize retaining walls for overall stability due to design seismic events may not be practical for walls placed on or near large marginally stable landslide areas. In general, if the placement of a wall within a marginally stable landslide area (i.e., marginally stable for static conditions) has only a minor effect on the stability of the landslide, the state reserves the right to not design the wall to prevent global instability of the landslide during the design seismic event.

6.1.2.2 Maximum Considered Depth for Liquefaction

When evaluating liquefaction potential and its impacts to transportation facilities, the maximum considered liquefaction depth shall be limited to 80 feet. The reasons for this limitation are as follows:

Limits of Simplified Procedures. The simplified procedures used to assess liquefaction potential are based on historical databases of liquefied sites with shallow liquefaction (less than 50 feet). Thus, these methodologies have not been calibrated to evaluate deep liquefaction. In addition, the simplified equation used to estimate the earthquake induced cyclic shear stress ratio (CSR) is based on a soil profile stiffness coefficient, r_d, that is highly variable at depth. At shallow depth (15 feet), r_d ranges from about 0.94 to 0.98. As depth increases, r_d becomes more variable ranging, for example, from 0.40 to 0.80 at a depth of 65 feet. The uncertainty regarding the coefficient r_d and lack of verification of the procedures used to predict liquefaction at depth, limit the depth at which these simplified procedures should be used.

Lack of Verification for More Rigorous Approaches. Several non-linear, effective stress analysis programs have been developed by researchers and can be used to estimate liquefaction potential at depth. However, there has been little field verification of the ability of these programs to predict liquefaction at depth because there are few well documented sites with deep liquefaction.

Decreasing Impact with Depth. Most of the effects of liquefaction decrease as the depth of the liquefiable layer increases. For example, the effects of a soil layer liquefying between depths of 80 and 90 feet will generally be much less severe than those of a layer between the depths of 10 and 20 feet.

Difficulties Mitigating for Deep Liquefaction. The geotechnical engineering profession has little experience with mitigation of liquefaction hazards at large depths, and virtually no field case histories on which to reliably verify the effectiveness of mitigation techniques for very deep liquefaction mitigation. In practicality, the costs to mitigate liquefaction by either ground improvement or designing the structure to tolerate the impacts of very deep liquefaction are excessive and not cost effective for most structures.

6.1.3 Governing Design Specifications and Additional Resources

The specifications applicable to seismic design of a given project depend upon the type of facility.

For bridge, roadway, retaining wall, and related transportation facility infrastructure design, seismic design criteria in the AASHTO LRFD Bridge Design Specifications and the WSDOT LRFD Bridge Design Manual (BDM), in addition to the requirements in the WSDOT Geotechnical Design Manual herein (GDM), shall be used. The BDM and GDM provide specific application of the AASHTO specifications to WSDOT design policy and practice.

For seismic design of new buildings and non-roadway infrastructure, the 2003 International Building Code (IBC) (**International Code Council, 2002**) should be used.

In addition to the above mentioned design specifications, geotechnical designers may utilize several resources that are available for geotechnical earthquake engineering. A brief description of these additional references are as follows:

FHWA Geotechnical Engineering Circular No. 3 (Kavazanjian, et al., 1997). This FHWA document provides design guidance for geotechnical earthquake engineering for highways. Specifically, this document provides guidance on earthquake fundamentals, seismic hazard analysis, ground motion characterization, site characterization, seismic site response analysis, seismic slope stability, liquefaction, and seismic design of foundations and retaining walls. The document also includes design examples for typical geotechnical earthquake engineering analyses.

NCHRP Report 472. The National Cooperative Highway Research Program Report 472 (2002), "Comprehensive specifications for the Seismic Design of Bridges", is a report containing the findings of a study completed to develop recommended specifications for seismic design of highway bridges. The report covers topics including design earthquakes and performance objectives, foundation design, liquefaction hazard assessment and design, and seismic hazard representation. Of particular interest, this document contains a case-study on liquefaction assessment of a hypothetical bridge in Washington State including estimating lateral spread induced loads on the bridge. Geotechnical seismic provisions in future versions of the AASHTO Specifications will likely be based, in-part, on the recommendations presented in NCHRP 472.

United States Geological Survey (USGS) Website. The USGS National Hazard Mapping Project website is a valuable tool for characterizing the seismic hazard for a specific site. The website allows the user to identify the peak ground acceleration (PGA) on bedrock and spectral acceleration ordinates at periods of 0.2, 0.3 and 1 second for risk levels of 2, 5 and 10 percent probabilities of exceedance (PE) in 50 years. The website also provides interactive deaggregation of a site's probabilistic seismic hazard. The deaggregation is useful in understanding the contribution of earthquakes of varying magnitude and distance to the seismic hazard at a site. The website address is http://eqhazmaps.usgs.gov/.

Geotechnical Earthquake Engineering Textbook. Professor Steven L. Kramer's textbook titled *Geotechnical Earthquake Engineering* (**Kramer, 1996**) provides a wealth of information to geotechnical engineers for seismic design. The textbook provides a comprehensive summary of seismic hazards, seismology and earthquakes, strong ground motion, seismic hazard analysis, wave propagation, dynamic soil properties, ground response analysis, design ground motions, liquefaction, seismic slope stability, seismic design of retaining walls, and ground improvement.

6.2 Geotechnical Seismic Design Considerations

6.2.1 Overview

The geotechnical designer has four broad options available for seismic design. They are:

- Use specification/code based hazard (**WSDOT GDM Section 6.3.1**) with specification/code based response (**WSDOT GDM Section 6.3.2**)
- Use specification/code based hazard (**WSDOT GDM Section 6.3.1**) with site specific response (**WSDOT GDM Appendix 6-A**)
- Use site specific hazard (**WSDOT GDM Appendix 6-A**) with specification/code based response (**WSDOT GDM Section 6.3.2**)
- Use site specific hazard (**WSDOT GDM Appendix 6-A**) with site specific response (**WSDOT GDM Appendix 6-A**)

Geotechnical parameters required for seismic design depend upon the type and importance of the structure, the geologic conditions at the site, and the type of analysis to be completed. For most structures, specification based design criteria appropriate for the site's soil conditions may be all that is required. Unusual, critical, or important structures may require more detailed structural analysis requiring additional geotechnical parameters. Finally, site conditions may require detailed geotechnical evaluation to quantify geologic hazards.

6.2.2 Site Characterization

As with any geotechnical investigation, the goal is to characterize the site soil conditions and determine how those conditions will affect the structures or features constructed when seismic events occur. In order to make this assessment, the geotechnical designer should review and discuss the project with the structural engineer, as seismic design is a cooperative effort between the geotechnical and structural engineering disciplines. The geotechnical designer should do the following as a minimum:

- Identify performance criteria (e.g., collapse prevention, limiting settlements, target safety factors, components requiring seismic design, etc.) and design risk levels (e.g., 10 percent PE in 50 years).
- Identify potential geologic hazards, areas of concern (e.g. soft soils), and potential variability of local geology.
- Identify the method by which risk-compatible ground motion parameters will be established (specification/code, deterministic, probabilistic, or a hybrid).

- Identify engineering analyses to be performed (e.g. site specific seismic response analysis, liquefaction susceptibility, lateral spreading/slope stability assessments).
- Identify engineering properties required for these analyses.
- Determine methods to obtain parameters and assess the validity of such methods for the material type.
- Determine the number of tests/samples needed and appropriate locations to obtain them.

It is assumed that the basic geotechnical investigations required for design have been or will be conducted as described in **WSDOT GDM Chapters 2, 5** and the individual project element chapters (e.g., **WSDOT GDM Chapter 8** for foundations, **WSDOT GDM Chapter 15** for retaining walls, etc.). Typically, the subsurface data required for seismic design is obtained concurrently with the data required for design of the project (i.e., additional exploration for seismic design over and above what is required for foundation design is typically not necessary). However, the exploration program may need to be adjusted to obtain the necessary parameters for seismic design. For instance, a seismic cone might be used in conjunction with a CPT if shear wave velocity data is required. Likewise, if liquefaction potential is a significant issue, mud rotary drilling with SPT sampling should be used. Hollow-stem auger drilling and non-standard samplers shall not be used to collect data used in liquefaction analysis and mitigation design.

The goal of the site characterization for seismic design is to develop the subsurface profile and soil property information needed for seismic analyses. Soil parameters generally required for seismic design include:

- Initial dynamic shear modulus at small strains or shear wave velocity;
- Equivalent viscous damping ratio;
- Shear modulus reduction and equivalent viscous damping characteristics as a function of shear strain;
- Cyclic shear strength parameters (peak and residual); and
- Liquefaction resistance parameters.

Table 6-1 provides a summary of site characterization needs and testing considerations for geotechnical/ seismic design.

Geotechnical Issues	Engineering Evaluations	Required Information for Analyses	Field Testing	Laboratory Testing
Site Response	• source characterization and attenuation • site response spectra • time history	• subsurface profile (soil, groundwater, depth to rock) • shear wave velocity • bulk shear modulus for low strains • relationship of shear modulus with increasing shear strain • equivalent viscous damping ratio with increasing shear strain • Poisson's ratio • unit weight • relative density • seismicity (PGA, design earthquakes)	• SPT • CPT • seismic cone • geophysical testing (shear wave velocity) • piezometer	• cyclic triaxial tests • Atterberg Limits • specific gravity • moisture content • unit weight • resonant column • cyclic direct simple shear test • torsional simple shear test
Geologic Hazards Evaluation (e.g. liquefaction, lateral spreading, slope stability)	• liquefaction susceptibility • liquefaction induced settlement • settlement of dry sands • lateral spreading • slope stability and deformations	• subsurface profile (soil, groundwater, rock) • shear strength (peak and residual) • unit weights • grain size distribution • plasticity characteristics • relative density • penetration resistance • shear wave velocity • seismicity (PGA, design earthquakes) • site topography	• SPT • CPT • seismic cone • Becker penetration test • vane shear test • piezometers • geophysical testing (shear wave velocity)	• soil shear tests • triaxial tests (including cyclic) • grain size distribution • Atterberg Limits • specific gravity • organic content • moisture content • unit weight

Input for Structural Design	• shallow foundation springs • P-y data for deep foundations • down-drag on deep foundations • residual strength • lateral earth pressures • lateral spreading/ slope movement loading • post earthquake settlement	• subsurface profile (soil, groundwater, rock) • shear strength (peak and residual) • seismic horizontal earth pressure coefficients • shear modulus for low strains or shear wave velocity • relationship of shear modulus with increasing shear strain • unit weight • Poisson's ratio • seismicity (PGA, design earthquake) • site topography	• CPT • SPT • seismic cone • piezometers • geophysical testing (shear wave velocity) • vane shear test	• triaxial tests • soil shear tests • unconfined compression • grain size distribution • Atterberg Limits • specific gravity • moisture content • unit weight • resonant column • cyclic direct simple shear test • torsional simple shear test

Table 6-1 Summary of site characterization needs and testing considerations for seismic design (adapted from Sabatini, et al., 2002).

WSDOT GDM Chapter 5 covers the requirements for how the results from the field investigation, the field testing, and the laboratory testing program are to be used separately or in combination to establish properties for design. For routine designs, in-situ field measurements or laboratory testing for parameters such as the dynamic shear modulus at small strains, equivalent viscous damping, shear modulus and damping ratio characteristics versus shear strain, and residual shear strength are generally not obtained. Instead, correlations based on index properties may be used in lieu of in-situ or laboratory measurements for routine design to estimate these values.

If correlations are used to obtain seismic soil design properties, the following correlations should be used:

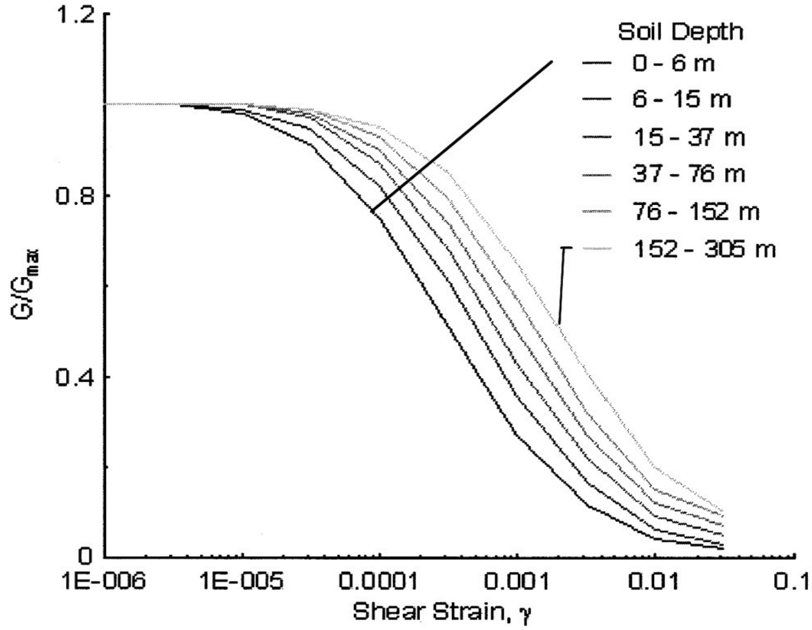
- **Table 6-2**, which presents correlations for estimating initial shear modulus based on relative density, penetration resistance or void ratio.
- **Figure 6-1**, which presents shear modulus reduction curves and equivalent viscous damping ratio for sands as a function of shear strain and depth.
- **Figures 6-2 and 6-3**, which present shear modulus reduction curves and equivalent viscous damping ratio, respectively, as a function of cyclic shear strain and plasticity index for fine grained soils
- **Figure 6-4**, which presents a chart for estimating equivalent undrained residual shear strength for liquefied soils as a function of SPT blowcounts.

Reference	Correlation	Units(1)	Limitations
Seed et al. (1984)	$G_{max} = 220\ (K_2)_{max}\ (\sigma'_m)^{½}$ $(K_2)_{max} = 20(N_1)_{60}^{1/3}$	kPa	$(K_2)_{max}$ is about 30 for very loose sands and 75 for very dense sands; about 80 to 180 for dense well graded gravels; Limited to cohesionless soils
Imai and Tonouchi (1982)	$G_{max} = 15{,}560\ N_{60}^{0.68}$	kPa	Limited to cohesionless soils
Mayne and Rix (1993)	$G_{max} = 99.5(P_a)^{0.305}(q_c)^{0.695}/(e_0)^{1.13}$	kPa(2)	Limited to cohesive soils; P_a = atmospheric pressure

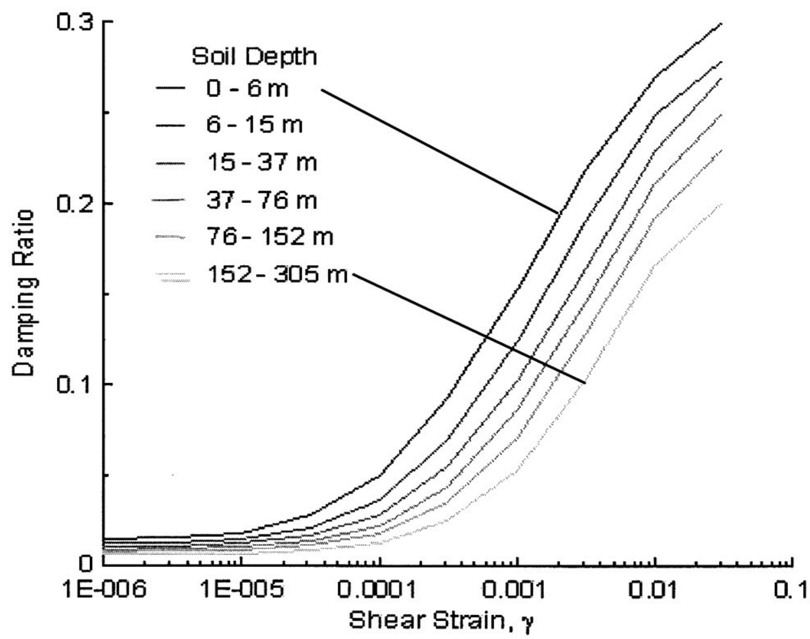
Notes: (1) 1 kPa = 20.885 psf

(2) P_a and q_c in kPa

Table 6-2 Correlations for estimating initial shear modulus (Kavazanjian, et al., 1997).



Shear Modulus Reduction Curves



Damping Ratio Curves

Figure 6-1 Shear modulus reduction and damping ratio curves for sand (EPRI, 1993).

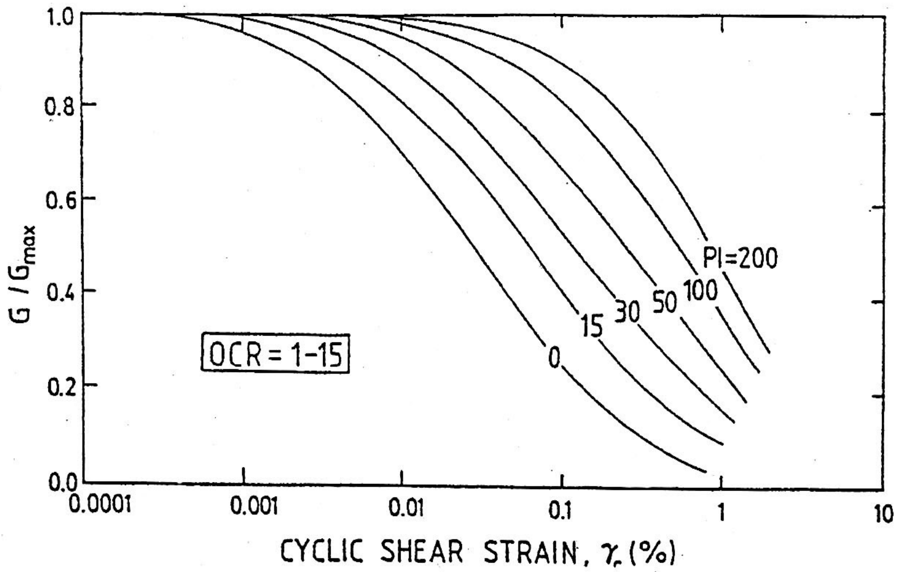


Figure 6-2 Shear modulus reduction curves for fine grained soils (Vucetic and Dobry, 1991).

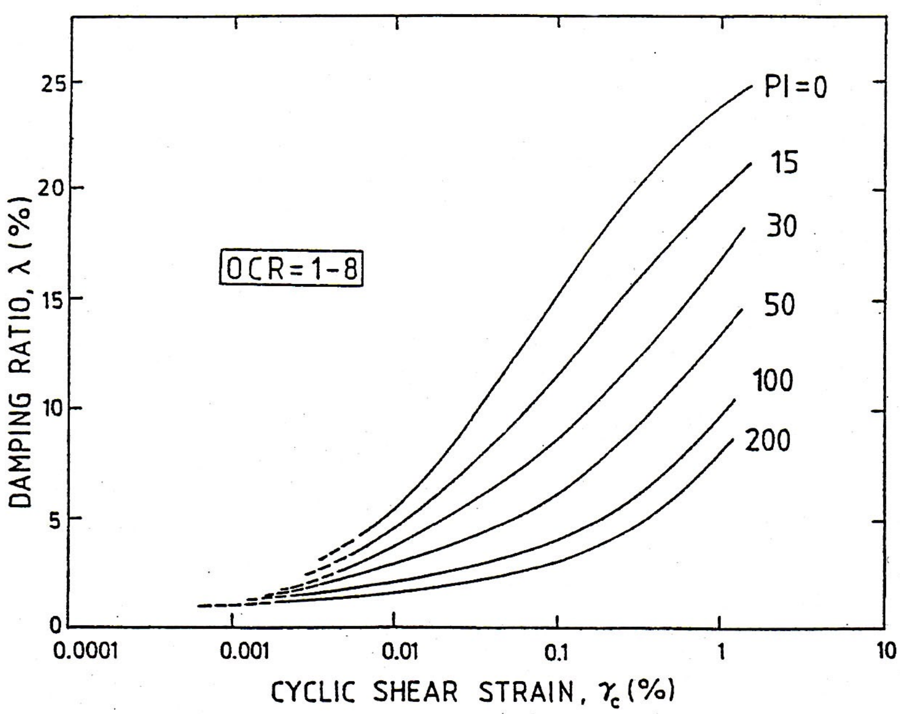


Figure 6-3 Equivalent viscous damping ratio for fine grained soils (Vucetic and Dobry, 1991).

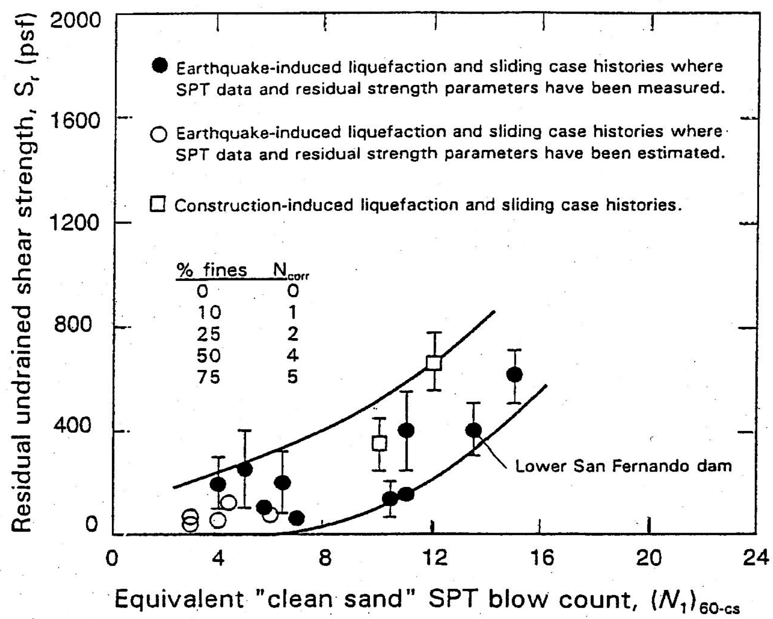


Figure 6-4 Equivalent undrained residual shear strength for liquefied soils as a function of SPT blowcounts (Seed and Harder, 1990).

6.2.3 Information for Structural Design

The geotechnical designer shall recommend a design ground motion, and shall evaluate geologic hazards for the project. In addition, the geotechnical designer should evaluate the site and soil conditions to the extent necessary to provide the following input for structural design:

- Foundation spring values for dynamic loading.
- Earthquake induced earth pressures for retaining structures and below grade walls.
- Impacts of seismic geologic hazards including fault rupture, liquefaction, lateral spreading and slope instability on infrastructure.
- Options to mitigate seismic geologic hazards, such as ground improvement.

6.3 Design Code Based Seismic Hazard and Site Response

For most projects, design code (specification) based seismic hazard and specification based site response are appropriate and should be used. Critical facilities, or projects where geologic conditions may result in un-conservative results if the generalized code response spectra is used, may require more detailed analysis such as probabilistic seismic hazard analysis, deterministic seismic hazards analysis, and/or site specific response analysis. Site specific hazard and response analysis is discussed further in **WSDOT GDM Appendix 6-A**.

6.3.1 Determination of Seismic Hazard Level

All non-critical transportation structures (e.g., bridges, pedestrian bridges, walls, and WSF terminal structures) shall be designed for no-collapse based on a risk level of 10 percent PE in 50 years (an approximately 475 year recurrence interval). **Figure 6-5** shall be used to estimate the PGA on bedrock for WSDOT transportation facilities, unless a site specific seismic hazard evaluation is conducted in accordance with **WSDOT GDM Appendix 6-A**. The PGA on bedrock contours in **Figure 6-5** are based on information published by the USGS National Seismic Hazards Mapping Project (**USGS, 2002**) and has been modified such that a minimum acceleration of 0.2g is used west of the Cascade Crest (west of MP 162 on SR 20; west of MP 65 on SR2; west of MP 52 on SR 90; west of MP 69 on SR 410; west of MP 151 on SR 12; and west of MP 63 on SR 14). Site response shall be in accordance with the AASHTO LRFD Bridge Design Specifications unless a site specific response analysis is performed as discussed in **WSDOT GDM Appendix 6-A**.

All critical transportation structures (e.g., bridges, pedestrian bridges, walls, and WSF terminal structures) shall be designed based on a risk level of 2 percent PE in 50 years (an approximately 2,500 year recurrence interval). For critical structures, the 2002 PGA on bedrock map for 2 percent PE in 50 years from the USGS National Seismic Hazards Mapping Project shall be used to estimate the acceleration coefficient unless a site specific seismic hazard evaluation is conducted in accordance with **WSDOT GDM Appendix 6-A**. Site response shall be in accordance with the AASHTO LRFD Bridge Design Specifications unless a site specific response analysis is performed as discussed in **WSDOT GDM Appendix 6-A**.

Interpolation between contours in **Figure 6-5** and the USGS 2 percent PE in 50 year map should be used when establishing the PGA on bedrock for a project.

For buildings, restrooms, shelters, and covered walkways, specification based seismic design parameters required by the 2003 IBC should be used. The seismic design requirements of the 2003 IBC are based on a risk level of 2 percent PE in 50 years. The 2 percent PE in 50 years risk level corresponds to the maximum considered earthquake. The 2003 IBC identifies procedures to develop a maximum considered earthquake acceleration response spectrum, and defines the design response spectrum as two-thirds of the value of the maximum considered earthquake acceleration response spectrum. The resulting design response spectrum is similar to the 10 percent PE in 50 years risk level (i.e., **Figure 6-5**) for much of Washington State. Site response shall be in accordance with the 2003 IBC. As is true for transportation structures, for critical or unique structures or for sites characterized as soil profile Type F (thick sequence of soft soils or liquefiable soils), site specific response analysis may be required as discussed in **WSDOT GDM Appendix 6-A**.

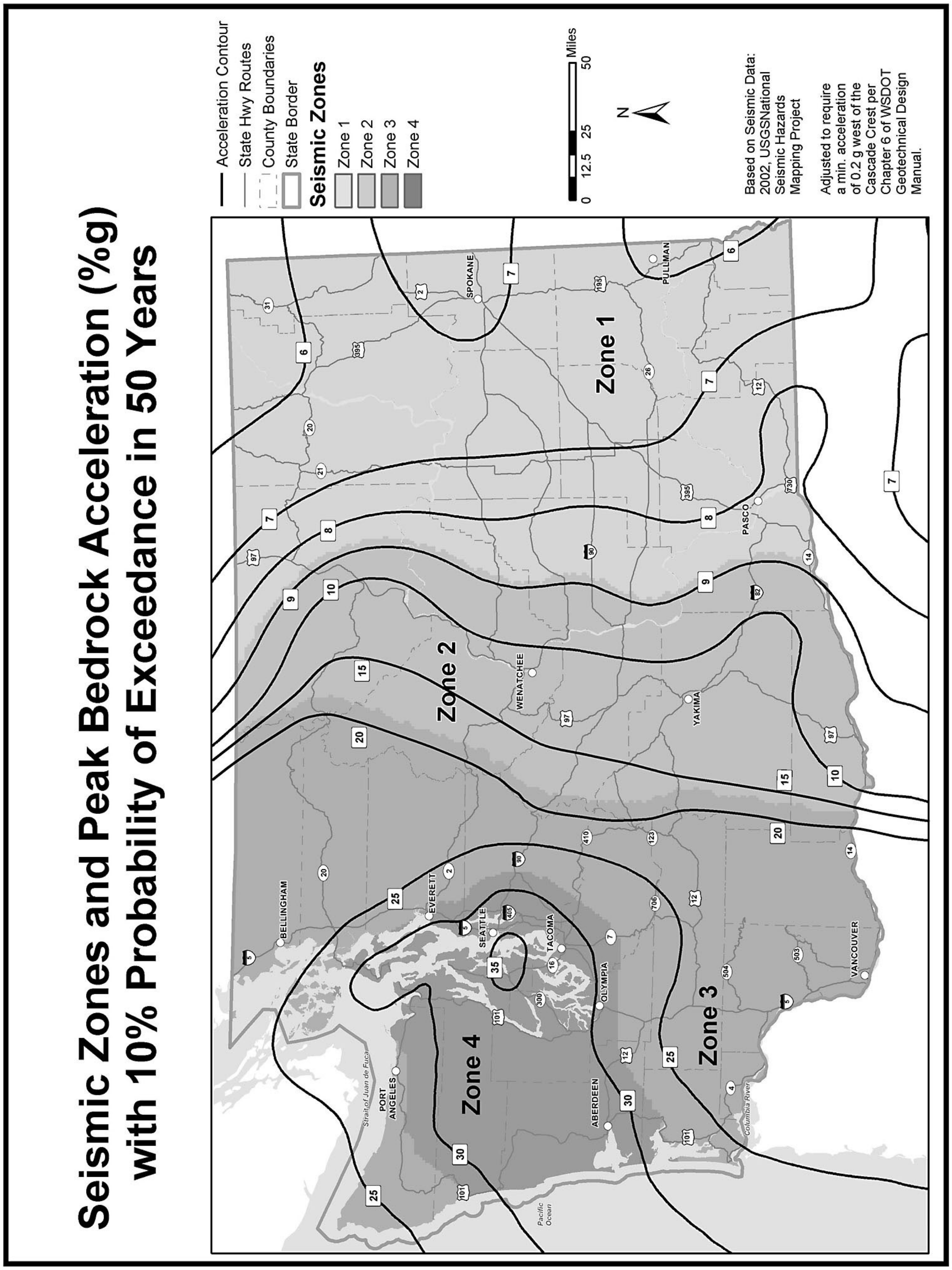


Figure 6-5 Peak ground acceleration/values on bedrock for seismic design in Washington State based on a risk level of 10 percent probability of exceedance in 50 years (adapted from USGS 2002, modified to require minimum peak acceleration of 0.2g west of Cascade crest).

6.3.2 2004 AASHTO Site Response

The AASHTO LRFD Bridge Design Specifications require that site effects be included in determining seismic loads for design of bridges. Depending upon the site soil conditions, one of four Site Coefficient (S) values is determined by the geotechnical designer. Table 3.10.5.1-1 of the AASHTO LRFD Bridge Design Specifications presents the value of the Site Coefficient for Soil Profile Types I through IV. The Site Coefficient, S, is used to account for the effect of site conditions on the elastic seismic response coefficient.

Once the Site Coefficient, S, is determined, the acceleration coefficient (PGA on bedrock) determined in **WSDOT GDM Section 6.3.1 or WSDOT GDM Appendix 6-A** is multiplied by S, and the elastic seismic response coefficient can be determined by the structural engineer for each mode of vibration of interest using Equation 3.10.6.1-1 in the AASHTO LRFD Bridge Design Specifications. Alternatively, the elastic seismic response spectrum can be computed for a range of periods as described in Section C3.10.6.1 of the AASHTO LRFD Bridge Design Specifications.

The AASHTO LRFD Bridge Design Specifications do not specifically require that a site specific seismic response analyses be completed for sites where liquefaction is anticipated during a design earthquake. Judgment should be applied to select an appropriate site coefficient. In general, soil profile Type III or IV may be selected to model the liquefaction case, or site specific response spectra may be developed. The decision to complete a site specific seismic response analysis where liquefaction is anticipated should be made by the geotechnical designer based on the site's geology and characteristics of the bridge being designed.

6.3.3 2003 IBC for Site Response

The 2003 IBC, Sections 1613 through 1615, provides procedures to estimate the earthquake loads for the design of buildings and similar structures. Earthquake loads per the 2003 IBC are defined by acceleration response spectra, which can be determined through the use of the 2003 IBC general response spectrum procedures or through site-specific procedures.

The general response spectrum per the 2003 IBC utilizes mapped Maximum Considered Earthquake (MCE) spectral response accelerations at short periods (S_S) and at 1-second (S_1) to define the seismic hazard at a specific location in the United States. The spectral accelerations presented on the 2003 IBC MCE maps are consistent with the 2 percent PE in 50 year risk level.

The intent of the 2003 IBC MCE is to reasonably account for the maximum possible earthquake at a site, and to preserve life safety and prevent collapse of the building. The 2003 IBC defines a Design Earthquake response spectrum as two-thirds of the MCE response spectrum. The Design Earthquake is used to establish the design earthquake loading of the portions of the structure not governed by collapse prevention under the MCE loading condition.

The 2003 IBC uses the seven site classes, Site Class A through Site Class F, to account for the effects of soil conditions on site response. Table 1615.1.1 of the 2003 IBC provides a summary of the site class definitions. The geotechnical designer should identify the appropriate Site Class for the site.

Once the Site Class and mapped values of S_s and S_1 are determined, values of the Site Coefficients F_a and F_v (site response modification factors) can be determined from Table 1615.1.2(1) of the 2003 IBC. The Site Coefficients and the mapped spectral accelerations S_s and S_1 can then be used to define the MCE response spectrum and the design response spectrum. The F_a values can also be used to estimate the PGA at the ground surface by multiplying the PGA on bedrock by the F_a value.

For sites where Site Class F soils are present, the 2003 IBC requires that a site-specific geotechnical investigation and dynamic site response analysis be completed, see **WSDOT GDM Appendix 6-A**. Dynamic site response analysis may not be required for liquefiable soil sites for structures with predominate periods of vibration less than 0.5 seconds.

6.3.4 Bedrock versus Ground Surface Acceleration

Amplification factors to account for the presence of soil over the bedrock with regard to the estimation of peak ground acceleration (PGA) are directly incorporated into the development of the standard response spectra for structural design of bridges and similar structures in the AASHTO LRFD Bridge Design Specifications and for the structural design of buildings and non-transportation related structures in the 2003 IBC. Additional amplification factors should not be applied to peak bedrock accelerations (PBA) when code based response spectra are used. However, amplification factors should be applied to PBA to determine PGA for liquefaction assessment and for the estimation of seismic earth pressures and inertial forces for retaining wall and slope design. For liquefaction assessment and retaining wall and slope design, the amplification factors presented in **Table 6-3** (**Stewart et al., 2003**) should be used, unless a site specific evaluation of ground response conducted in accordance with **GDM Appendix 6-A** is performed.

Site Class	General Material Type	Shear Wave Velocity (V_s) m/s	Shear Wave Velocity (V_s) ft/s	N_{ave} bpf	S_u (ave) kPa	S_u (ave) psf
A	Hard Rock	>1500	>5000	--	--	--
B	Rock	760 to 1500	2500 to 5000	--	--	--
C	Very Dense Soil & Soft Rock	360 to 760	1200 to 2500	>50	>100	>2000
D	Stiff Soil	180 to 360	600 to 1200	15 to 50	50 to 100	1000 to 2000
E	Soil	<180	<600	<15	<50	<1000

Amplification Factor (F) for PGA F=exp[a+b(ln(PGA))] Seismological Society of America Vol. 93, No. 1, pp. 332-352. Stewart et al.

Site Class	Const.		Peak Bedrock Acceleration -->																
	a	b	0.01	0.05	0.10	0.12	0.14	0.16	0.18	0.20	0.22	0.24	0.26	0.28	0.30	0.32	0.34	0.36	0.38
A	--	--	*0.80*	*0.80*	*0.80*	*0.80*	*0.80*	*0.80*	*0.80*	*0.80*	*0.80*	*0.80*	*0.80*	*0.80*	*0.80*	*0.80*	*0.80*	*0.80*	*0.80*
B	0.09	0.05	0.87	0.94	0.98	0.98	0.99	1.00	1.00	1.01	1.01	1.02	1.02	1.03	1.03	1.03	1.04	1.04	1.04
C	-0.06	-0.05	1.19	1.09	1.06	1.05	1.04	1.03	1.03	1.02	1.02	1.01	1.01	1.00	1.00	1.00	0.99	0.99	0.99
D	0.08	-0.07	1.50	1.34	1.27	1.26	1.24	1.23	1.22	1.21	1.20	1.20	1.19	1.18	1.18	1.17	1.17	1.16	1.16
E	-0.60	-0.50	5.49	2.45	1.74	1.58	1.47	1.37	1.29	1.23	1.17	1.12	1.08	1.04	1.00	0.97	0.94	0.91	0.89

Note: NEHRP Factor for Site A is 0.8 for all accelerations. Stewart did not provide factors for Site A. Use NEHRP values.

Factored PGA using (F)

Site Class	PGA from Map -->																
	0.01	0.05	0.10	0.12	0.14	0.16	0.18	0.20	0.22	0.24	0.26	0.28	0.30	0.32	0.34	0.36	0.38
A	0.01	0.04	0.08	0.10	0.11	0.13	0.14	0.16	0.18	0.19	0.21	0.22	0.24	0.26	0.27	0.29	0.30
B	0.01	0.05	0.10	0.12	0.14	0.16	0.18	0.20	0.22	0.24	0.27	0.29	0.31	0.33	0.35	0.37	0.40
C	0.01	0.05	0.11	0.13	0.15	0.17	0.18	0.20	0.22	0.24	0.26	0.28	0.30	0.32	0.34	0.36	0.38
D	0.01	0.07	0.13	0.15	0.17	0.20	0.22	0.24	0.26	0.29	0.31	0.33	0.35	0.38	0.40	0.42	0.44
E	0.05	0.12	0.17	0.19	0.21	0.22	0.23	0.25	0.26	0.27	0.28	0.29	0.30	0.31	0.32	0.33	0.34

Table 6-3 Amplification Factors for Bedrock PGA based on NEHRP Site Class (Stewart et al., 2003).

6.3.5 Earthquake Magnitude

Assessment of liquefaction and lateral spreading require an estimate of the earthquake magnitude. For routine design, a default moment magnitude of 7.0 should be used for western Washington and 6.0 for eastern Washington, except within 30 miles of the coast where Cascadia Subduction zone events contribute significantly to the seismic hazard. In that case, the geotechnical designer should use a moment magnitude of 8.0. Note that these default magnitudes are intended for use in liquefaction and lateral spreading analysis only and should not be used for development of the design ground motion parameters. Additional discussion and guidance regarding this issue is provided in **WSDOT GDM Appendix 6-A**.

6.4 Input for Structural Design

6.4.1 Foundation Springs

Structural dynamic response analyses incorporate the foundation stiffness into the dynamic model of the structure to capture the effects of soil structure interaction. The foundation stiffness is typically represented as a system of equivalent springs placed in a foundation stiffness matrix. The typical foundation stiffness matrix incorporates a set of six springs, namely a vertical spring, horizontal springs in the orthogonal plan dimensions, rocking about each horizontal axis, and torsion around the vertical axis.

The primary parameters for calculating the individual springs are the foundation type (shallow spread footings or deep foundations), foundation geometry, and dynamic soil shear modulus. The dynamic soil shear modulus is a function of the shear strain (foundation displacement), so determining the appropriate foundation springs can be an iterative process.

6.4.1.1 Shallow Foundations

For evaluating shallow foundation springs, the WSDOT Bridge and Structures Office requires values for the dynamic shear modulus, G, Poisson’s ratio, and the unit weight of the foundation soils. The maximum, or low-strain, shear modulus can be estimated using index properties and the correlations presented in **Table 6-2**. Alternatively, the maximum shear modulus can be calculated using **Equation 6-1** below, if the shear wave velocity is known:

$$G_{\max} = \frac{\gamma}{g}\left(V_s\right)^2 \qquad (6\text{-}1)$$

where:

G_{max}	=	maximum dynamic shear modulus
γ	=	soil unit weight
V_s	=	shear wave velocity
g	=	acceleration due to gravity

The maximum dynamic shear modulus is associated with small shear strains (less than 0.0001 percent). As shear strain level increases, dynamic shear modulus decreases. At large cyclic shear strain (1 percent), the dynamic shear modulus approaches a value of approximately 10 percent of G_{max} (**Seed et al., 1986**). As a minimum, shear modulus values for 0.2 percent shear strain and 0.02 percent shear strain to simulate large and small magnitude earthquakes should be provided to the structural engineer. Shear modulus values at other shear strains could also be provided as needed for the design. Shear modulus values may be estimated using **Figures 6-1 and 6-2**. Alternatively, laboratory tests, such as the cyclic triaxial or resonant column tests, may be used to determine the shear modulus values. Poisson's Ratio can be estimated based on soil type, relative density/consistency of the soils, and correlation charts such as those presented in **GDM Chapter 5** or in the textbook, *Foundation Analysis and Design* (**Bowles, 1996**).

6.4.1.2 Deep Foundations

Lateral soil springs for deep foundations shall be determined in accordance with **GDM Chapter 8**.

6.4.2 Earthquake Induced Earth Pressures on Retaining Structures

The Mononobe-Okabe pseudo-static method shall be used to estimate the seismic lateral earth pressure, as specified in **WSDOT GDM Chapter 15**.

6.4.3 Downdrag Loads on Structures

Downdrag loads on foundations shall be determined in accordance with **WSDOT GDM Chapter 8**.

6.4.4 Lateral Spread / Slope Failure Loads on Structures

In general, there are two different approaches to estimate the lateral spread induced load on deep foundations systems—a displacement based method and a force based method. Displacement based methods are more prevalent in the United States. The force based approach has been specified in the Japanese codes and is based on case histories from past earthquakes, especially the pile foundation failures observed during the 1995 Kobe earthquake. Overviews of both approaches are presented below.

6.4.4.1 Displacement Based Approach

The recommended displacement based approach for evaluating the impact of liquefaction induced lateral spreading loads on deep foundation systems is presented in the NCHRP Report 472 titled "Comprehensive Specification for Seismic Design of Bridges" (**NCHRP, 2002**) and supporting documentation by (**Martin et al., 2002**). The general procedure is as follows:

Evaluate the Liquefaction Potential: Evaluate the liquefaction potential of the site for the design risk levels. Assign residual and reduced strength parameters to liquefied and partially liquefied soils layers.

Conduct Slope Stability Analyses: If liquefaction is predicted, conduct slope stability analyses using residual strength parameters for the liquefied soil layers and reduced strength parameters for partially liquefied soil layers. If the static factor of safety is less than unity, a flow failure is predicted. If the static factor of safety is greater than unity, conduct pseudo-static stability analyses to determine the yield acceleration K_y.

Check Zone of Influence: Assess whether or not the estimated failure surface could impact the bridge foundation system. If the bridge foundations are expected to be within the zone of influence, estimate the ground deformations.

Slope Deformations: For potential failure surfaces with static factors of safety less than 1.0 for post liquefaction conditions, flow failure is predicted and displacements are anticipated to be large. For potential failure surfaces with yield accelerations greater than zero (factor of safety greater than 1.0 for static conditions), estimate the maximum lateral spread induced displacements. Appropriate methods for estimating lateral displacements associated with flow failure and lateral spreading may include the empirical procedure developed by **Youd et al. (2002)**, dynamic runout modeling, or Newmark-type analyses. Newmark-type models should only be used to estimate displacements associated with lateral spreading if the static factor of safety is greater than 1.0 (See **Section 6.5.4**).

Induced Loads on Foundation Elements: Assess whether the soil will displace and flow around a stable foundation or whether foundation movement will occur in concert with the soil. This assessment requires a comparison between the estimated passive soil forces that can be exerted on the foundation and the ultimate resistance that can be provided by the structure.

The magnitudes of moment and shear induced in the foundations by the ground displacement can be estimated using soil-pile structure interaction programs, such as L-Pile or S-Shaft (see **WSDOT GDM Chapter 8** for additional discussion on L-Pile and S-Shaft). The process is to apply the assumed displacement field to the interface springs whose properties are represented by P-y curves. With older versions of L-Pile, the liquefied soil layers are typically modeled as a soft clay using the undrained residual strength of the liquefied soil (see **Figure 6-4**). L-Pile Plus version 5.0 includes P-y curves for liquefied sands that more accurately model the strain hardening behavior observed from liquefied soils. Partially liquefied soil layers are typically adjusted by reducing their friction angle. The strength parameters of non-liquefied layers above and/or below the liquefied zones are not reduced.

A similar approach can be used with the S-Shaft program, which is based on the Strain Wedge Model (see **WSDOT GDM Chapter 8** for additional information on the strain wedge model). S-Shaft program has an option built in to the program for estimating lateral spread loads on a single pile or shaft.

The estimated induced loads are then checked against the ability of the foundation system to resist those loads. The ultimate foundation resistance is based in part on the resistance provided by the portion of the pile/shaft embedded in non-liquefiable soils below the lateral spread zone and the structural capacity of the pile/shaft. Large pile deformations may result in plastic hinges forming in the pile/shaft. If foundation resistance is greater than that applied by the lateral spreading soil, the soil will flow around the structure. If the potential load applied by the soil is greater than the ultimate foundation system resistance, the pile/shaft is likely to move in concert with the soil. Also, the passive pressure generated on the pile cap by the spreading soil needs to be considered in the total load applied to the foundation system. In cases where a significant crust of non-liquefiable material may exist, the foundation is likely to continue to move with the soil. Since large-scale structural deformations may be difficult and costly to accommodate in design, mitigation of foundation subsoils will likely be required.

Similar approaches to those outlined above can be used to estimate loads that other types of slope failure may have on the bridge foundation system.

6.4.4.2 Force Based Approaches

A force based approach to assess lateral spreading induced loads on deep foundations is specified in the Japanese codes. The method is based on back-calculations from pile foundation failures caused by lateral spreading. The pressures on pile foundations are simply specified as follows:

- The liquefied soil exerts a pressure equal to 30 percent of the total overburden pressure (lateral earth pressure coefficient of 0.30 applied to the total vertical stress).
- Non-liquefied crustal layers exert full passive pressure on the foundation system.

Data from simulated earthquake loading of model piles in liquefiable sands in centrifuge tests indicate that the Japanese force method is an adequate design method **(Finn, 2004)**.

Another force-based approach to estimate lateral spreading induced foundation loads is to use a limit equilibrium slope stability program to determine the load the foundation must resist to achieve a target safety factor of 1.1. This force is distributed over the foundation in the liquefiable zone as a uniform stress. This approach may be utilized to estimate the forces that foundation elements must withstand if they are to act as shear elements stabilizing the slope. See **WSDOT GDM Section 6.5.3** for specific stability analysis procedures.

6.4.4.3 Mitigation Alternatives

The two basic options to mitigate the lateral spread induced loads on the foundation system are to design the structure to accommodate the loads or improve the ground such that the hazard does not occur.

Structural Options (design to accommodate imposed loads). The general structural approach to design for the hazard is outlined below.

<u>Step 1:</u> If the soil is expected to displace around the foundation element, the foundation is designed to withstand the passive force exerted on the foundation by the flowing soil. In this case, the maximum loads determined from the P-y springs for large deflections are applied to the pile/shaft, and the pile/shaft is evaluated using a soil structure interaction program similar to L-Pile or S-Shaft. The pile/shaft stiffness, strength, and embedment are adjusted until the desired structural response to the loading is achieved.

Note that it is customary to evaluate the lateral spread/slope failure induced loads independently from the inertial forces caused by the shaking forces (i.e. the shaking force loads and the lateral spread loads are typically not assumed to act concurrently). In most cases this is reasonable since peak vibration response is likely to occur in advance of maximum ground displacement, and displacement induced maximum shear and moments will generally occur at deeper depths than those from inertial loading.

<u>Step 2:</u> If the assessment indicates that movement of the foundation is likely to occur in concert with the soil, then the structure is evaluated for the maximum expected ground displacement. In this case the soil loads are generally not the maximum possible (loads at large displacements), but instead some fraction thereof. Again the P-y data for the soils in question are used to estimate the loading.

If the deformations determined in step 2 are beyond tolerable limits for structural design, the options are to a) re-evaluate the deformations based on the “pinning” or “doweling” action that foundations provide as they cross a potential failure plane (with consideration of the foundation strength; or b) re-design the foundation system to accommodate the anticipated loads. Simplified procedures for evaluating the available resistance to slope movements provided by the foundation “pinning” action are presented in (**NCHRP, 2002**) and (**Martin, et al., 2002**) and require knowledge of the plastic moment and location of plastic hinges in the foundation elements; this information should be provided by the bridge engineer or structural consultant. The concept of considering a plastic mechanism or hinging in the piles/shafts is tantamount to accepting foundation damage.

With input from the structural engineer regarding “pinning” resistance provided by the foundation system, recalculate the estimated displacement based on the revised resistance levels. If the structure’s behavior is acceptable under the revised displacement estimate, the design for liquefaction induced lateral spreading is complete. If the performance is not acceptable, then the foundation system should be redesigned or ground improvement should be considered.

Ground Improvement. It is often cost prohibitive to design the bridge foundation system to resist the loads imposed by liquefaction induced lateral loads, especially if the depth of liquefaction extends more than about 20 feet below the ground surface and if a non-liquefied crust is part of the failure surface. Ground improvement to mitigate the liquefaction hazard is the likely alternative if it is not practical to design the foundation system to accommodate the lateral loads.

The primary ground improvement techniques to mitigate liquefaction fall into three general categories, namely densification, altering the soil composition, and enhanced drainage. A general discussion regarding these ground improvement approaches is provided below. **WSDOT GDM Chapter 11**, Ground Improvement, of this manual should be reviewed for a more detailed discussion regarding the use of these techniques.

<u>Densification and Reinforcement:</u> Ground improvement by densification consists of sufficiently compacting the soil such that it is no longer susceptible to liquefaction during a design seismic event. Densification techniques include vibro-compaction, vibro-flotation, vibro-replacement (stone columns), deep dynamic compaction, blasting, and compaction grouting. Vibro-replacement and compaction grouting also reinforce the soil by creating columns of stone and grout, respectively. The primary parameters for selection include grain size distribution of the soils being improved, depth to groundwater, depth of improvement required, proximity to settlement/vibration sensitive infrastructure, and access constraints.

<u>Altering Soil Composition:</u> Altering the composition of the soil typically refers to changing the soil matrix so that it is no longer susceptible to liquefaction. Example ground improvement techniques include permeation grouting (either chemical or micro-fine cement), jet grouting, and deep soil mixing. These types of ground improvement are typically more costly than the densification/reinforcement techniques, but may be the most effective techniques if access is limited, construction induced vibrations must be kept to a minimum, and/or the improved ground has secondary functions, such as a seepage barrier or shoring wall.

<u>Drainage Enhancements:</u> By improving the drainage properties of soils susceptible to liquefaction, it may be possible to prevent the build-up of excess pore water pressures, and thus liquefaction. However, drainage improvement is not considered adequately reliable by WSDOT to prevent excess pore water pressure buildup due to liquefaction due to drainage path time for pore pressure to dissipate, and due to the potential for drainage structures to become clogged during installation and in service. In addition, with drainage enhancements some settlement is still likely. Therefore, drainage enhancements shall not be used as a means to mitigate liquefaction.

6.5 Seismic Geologic Hazards

The geotechnical designer shall evaluate seismic geologic hazards including fault rupture, liquefaction, lateral spreading, ground settlement, and slope instability. The risk associated with seismic geologic hazards shall be evaluated by the geotechnical designer.

6.5.1 Fault Rupture

Washington State is recognized as a seismically active region; however, only a relatively small number of active faults have been identified within the state. Thick sequences of recent geologic deposits, heavy vegetation, and the limited amount of instrumentally recorded events on identified faults are some of the factors that contribute to the difficulty in identifying active faults in Washington State. Considerable research is ongoing throughout Washington State to identify and characterize the seismicity of active faults, and new technology makes it likely that additional surface faults will be identified in the near future.

Figure 6-6 presents the earthquake faults considered to be potentially active. The following faults are explicitly included in the 2002 USGS probabilistic hazard maps:

- Seattle Fault Zone
- Southern Whidbey Island Fault
- Utsalady Fault
- Strawberry Point Fault
- Devils Mountain Fault
- Horse Heaven Hills Anticline
- Rattlesnake-Wallula Fault System
- Mill Creek Fault
- Saddle Mountains Fault
- Hite Fault System

The potential impacts of fault rupture include abrupt, large, differential ground movements and associated damage to structures that might straddle a fault, such as a bridge. WSDOT recognizes that due to the limited number of mapped active faults and the frequent presence of thick soil overburden, the ability to identify potential surface expressions of faulting is unreliable at this time. However, the potential for fault rupture should be evaluated and taken into consideration in the planning and design of new facilities.

6.5.2 Liquefaction

Liquefaction has been one of the most significant causes of damage to bridge structures during past earthquakes (**NCHRP, 2002**). Liquefaction can damage bridges and structures in many ways including:

- Modifying the nature of ground motion;
- Bearing failure of shallow foundations founded above liquefied soil;
- Liquefaction induced ground settlement;
- Lateral spreading of liquefied ground;
- Large displacements associated with low frequency ground motion;
- Increased earth pressures on subsurface structures;
- Floating of buoyant, buried structures; and
- Retaining wall failure.

Liquefaction refers to the significant loss of strength and stiffness resulting from the generation of excess pore water pressure in saturated cohesionless soils. Liquefaction can occur in gravel to silt size soils; however, it is most common in sands. **Kramer (1996)** provides a detailed description of liquefaction including the types of liquefaction phenomena, evaluation of liquefaction susceptibility, and the effects of liquefaction.

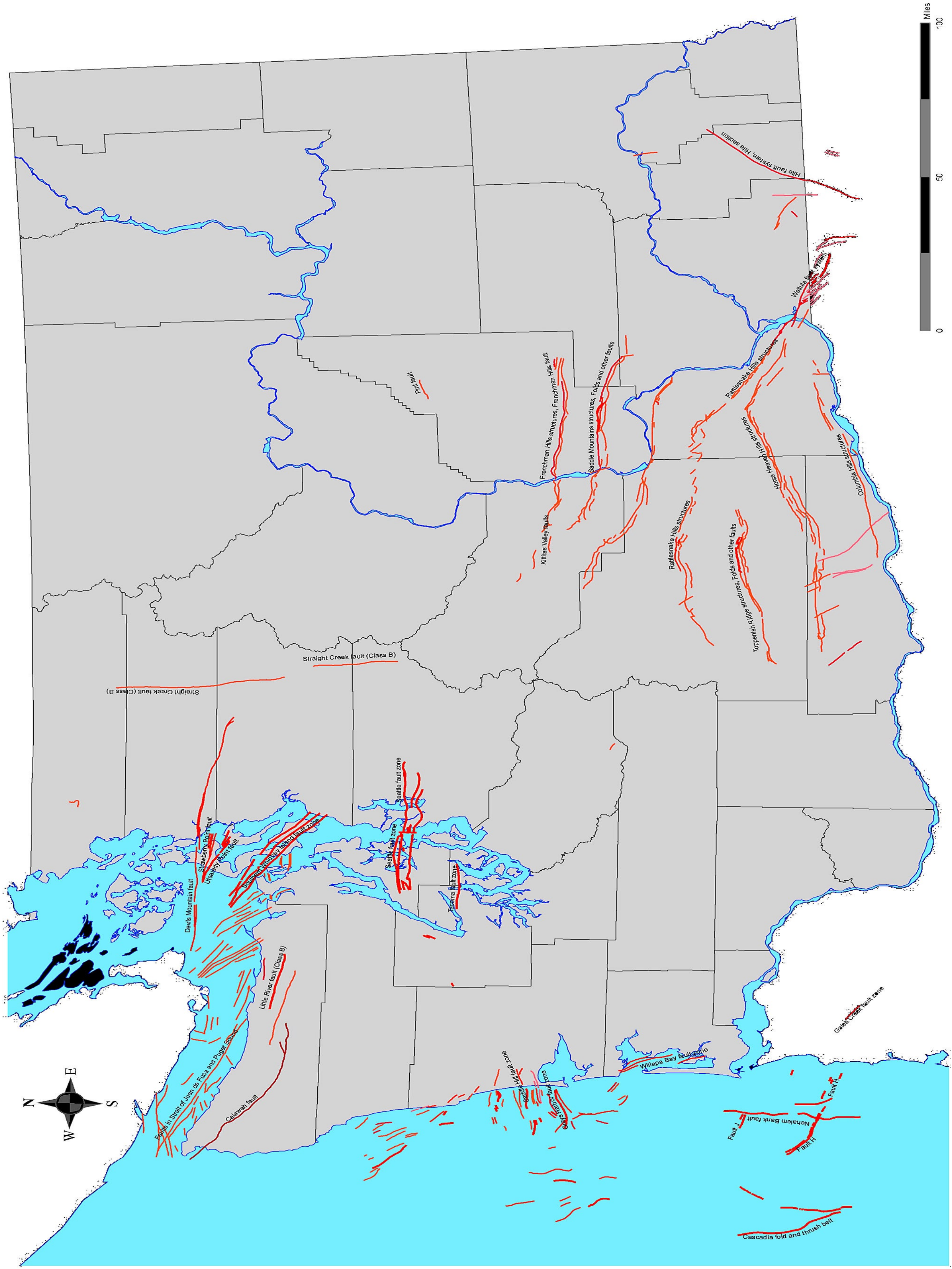


Figure 6-6 Earthquake Faults in Washington State.

Liquefaction hazard assessment includes identifying soils prone to liquefaction, evaluating whether the design earthquake loading will initiate liquefaction, and estimating the potential effects of liquefaction on the planned facility. The following sections provide an overview of liquefaction hazard assessment and its mitigation.

6.5.2.1 Methods to Evaluate Liquefaction Potential

Evaluation of liquefaction potential should be completed based on soil characterization using in-situ testing such as Standard Penetration Tests (SPT) and Cone Penetration Tests (CPT). Liquefaction potential may also be evaluated using shear wave velocity (V_S) testing and Becker Penetration Tests (BPT); however, these methods are not preferred and are used less frequently than SPT or CPT methods. V_S and BPT testing may be appropriate in soils difficult to test using SPT and CPT methods, such as gravelly soils. If the CPT method is used, SPT sampling and soil gradation testing shall still be conducted to obtain direct information on soil gradation parameters for liquefaction susceptibility assessment and input into the Simplified Method. Liquefaction susceptibility of silts can be evaluated using the Modified Chinese Criteria.

Once a preliminary screening is performed, liquefaction potential shall be evaluated using the Simplified Procedure. More rigorous, nonlinear, dynamic, effective stress computer models such as DESRA (**Lee et al., 1978**) may be used for site conditions or situations that are not modeled well by the Simplified Method, subject to the approval of the State Geotechnical Engineer. The Simplified Procedure was originally developed by **Seed and Idriss (1971)** and has been periodically modified and improved since. The Simplified Procedure is routinely used to evaluate liquefaction resistance in geotechnical practice.

Preliminary Screening. A detailed evaluation of liquefaction potential is not required if one or more of the following conditions occur at a site:

- The estimated maximum groundwater level at the site is determined to be deeper than 75 ft below the existing ground surface or proposed finished grade, which ever is deeper.
- The subsurface profile is characterized as having a minimum SPT resistance, corrected for overburden depth and hammer energy ($N1_{60}$), of 30 blows/ft, or a cone tip resistance q_c of more than 160 tsf, or if bedrock is present to the ground surface.
- The soil is clayey, as defined by the Modified Chinese Criteria described below.

If the site does not meet one of the conditions described above, a more detailed assessment of liquefaction shall be conducted.

Simplified Procedure. The paper titled "Liquefaction Resistance of Soils: Summary Report from the 1996 NCEER/NSF Workshops on Evaluation of Liquefaction Resistance of Soils" by **Youd et al., (2001)** provides a state of the practice summary of the Simplified Procedure for assessment of liquefaction susceptibility. This paper resulted from a 1996 workshop of liquefaction experts sponsored by the National Center for Earthquake Engineering Research and the National Science Foundation with the objective being to gain consensus on updates and augmentation of the Simplified Procedure.
Youd et al. (2001) provides procedures for evaluating liquefaction susceptibility using SPT, CPT, V_S, and BPT criteria.

The Simplified Procedure is based on comparing the cyclic resistance ratio (CRR) of a soil layer (i.e., the cyclic shear stress required to cause liquefaction) to the earthquake induced cyclic shear stress ratio (CSR). The resistance value is estimated based on empirical charts relating the resistance available to specific index properties (i.e. SPT, CPT, BPT or shear wave velocity values) and corrected to an equivalent magnitude of 7.5 using a magnitude scaling factor. **Youd et al. (2001)** provide the empirical liquefaction resistance charts for both SPT and CPT data to be used with the Simplified Method.

The earthquake induced CSR for the Simplified Method shall be estimated using **Equation 6-2**:

$$CSR = 0.65 \frac{A_{max}}{g} \frac{\sigma_o}{\sigma_o'} r_d \qquad (6\text{-}2)$$

Where
- A_{max} = peak ground acceleration accounting for site amplification effects
- g = acceleration due to gravity
- σ_o = initial total vertical stress at depth being evaluated
- σ_o' = initial effective vertical stress at depth being evaluated
- r_d = stress reduction coefficient

Note that A_{max} is the PGA times the acceleration due to gravity, since the PGA is actually an acceleration coefficient.

The factor of safety against liquefaction is defined by **Equation 6-3**:

$$FS_{liq} = CRR/CSR \qquad (6\text{-}3)$$

The SPT procedure has been most widely used and has the advantage of providing soil samples for fines content and gradation testing. The CPT provides the most detailed soil stratigraphy, is less expensive, can simultaneously provide shear wave velocity measurements, and is more reproducible. The use of both SPT and CPT procedures can provide a detailed liquefaction assessment for a site.

Where SPT data is used, sampling and testing shall be conducted in accordance with **WSDOT GDM Chapter 3**. In addition:

- Correction factors for borehole diameter, rod length and sampler liners should be used, where appropriate.
- Where gravels or cobbles are present, the use of short interval adjusted SPT N values may be effective for estimating the N values for the portions of the sample not affected by gravels or cobbles.
- Blowcounts obtained when sampling using Dames and Moore or modified California samplers shall not be used for liquefaction evaluations.

As discussed in **WSDOT GDM Section 6.1.2.2**, the limitations of the Simplified Procedure should be recognized. The Simplified Procedure was developed from empirical evaluations of field observations. Most of the case history data was collected from level to gently sloping terrain underlain by Holocene-age alluvial or fluvial sediment at depths less than 50 feet. Therefore, the Simplified Procedure is applicable to only these site conditions. Caution should be used for evaluating liquefaction potential at depths greater than 50 feet using the Simplified Procedure. In addition, the Simplified Procedure estimates the earthquake induced cyclic shear stress ratio based on a coefficient, r_d, that is highly variable at depth as discussed in **WSDOT GDM Section 6.1.2.2**.

As an alternative to the use of the r_d factor, to improve the assessment of liquefaction potential, especially at greater depths, equivalent linear or nonlinear site specific, one dimensional ground response analyses may be conducted to determine the maximum earthquake induced shear stresses at depth in the Simplified Method. For example, the linear total stress computer programs ProShake (**EduPro Civil Systems, 1999**) or Shake2000 (**Ordoñez, 2000**) may be used for this purpose.

Modified Chinese Criteria. The Modified Chinese Criteria should be used to assess the liquefaction susceptibility of fine "cohesive" soils. According to the Modified Chinese Criteria (**Finn et al., 1994**), fine-grained soils are considered potentially liquefiable if:

- The soil has less than 15 percent finer than 0.005mm;
- The soil has a Liquid Limit (LL) less than or equal to 35 percent; and
- The in-situ water content of the soil is greater than or equal to 90 percent of the LL.

Due to the ability to typically obtain higher quality undisturbed samples of fine-grained soils, laboratory cyclic triaxial shear testing may be used to evaluate the liquefaction susceptibility of finer grained soils in lieu of the Modified Chinese Criteria.

Nonlinear Effective Stress Method. An alternative to the Simplified Procedure for evaluating liquefaction susceptibility is to complete a nonlinear, effective stress site response analysis utilizing a computer code capable of modeling pore water pressure generation and dissipation. This is a more rigorous analysis that requires additional soil parameters.

The advantages with this method of analysis include liquefaction at depths greater than 50 feet, the effects of liquefaction and large shear strains on the ground motion, and the effects of higher accelerations that can be more reliably evaluated. In addition, seismically induced deformation can be estimated, and the timing of liquefaction and its effects on ground motion at and below the ground surface can be assessed.

Several non-linear, effective stress analysis programs developed by researchers can be used to estimate liquefaction susceptibility at depth. However, few of these programs are commercially available and being used by geotechnical designers at this time. In addition, there has been little verification of the ability of these programs to predict liquefaction at depths greater than 50 feet because there are few well documented sites of deep liquefaction.

Due to the research nature of these more sophisticated liquefaction assessment approaches, approval by the WSDOT State Geotechnical Engineer is required to use nonlinear effective stress methods for liquefaction evaluation.

6.5.2.2 Minimum Factor of Safety Against Liquefaction

Liquefaction hazards assessment and the development of hazard mitigation measures shall be conducted if the factor of safety against liquefaction (**Equation 6-3**) is less than 1.2. Liquefaction hazards to be assessed include settlement and related effects, and liquefaction induced instability (e.g., flow failure or lateral spreading).

6.5.2.3 Liquefaction Induced Settlement

Both dry and saturated deposits of loose granular soils tend to densify and settle during earthquake shaking. Settlement of unsaturated granular deposits is discussed in **WSDOT GDM Section 6.5.3**. Settlement of saturated granular deposits due to liquefaction shall be estimated using techniques based on the Simplified Procedure, or if nonlinear effective stress models are used to assess liquefaction in accordance with **WSDOT GDM Section 6.5.2.1**, such methods may also be used to estimate liquefaction settlement.

If the Simplified Procedure is used to evaluate liquefaction potential, liquefaction induced ground settlement of saturated granular deposits should be estimated using the procedures by **Tokimatsu and Seed (1987)** or **Ishihara and Yoshimine (1992)**. The **Tokimatsu and Seed (1987)** procedure estimates the volumetric strain as a function of earthquake induced CSR and corrected SPT blowcounts. The **Ishihara and Yoshimine (1992)** procedure estimates the volumetric strain as a function of factor of safety against liquefaction, relative density, and corrected SPT blowcounts or normalized CPT tip resistance. Example charts used to estimate liquefaction induced settlement using the Tokimatsu and Seed procedure and the Ishihara and Yoshimine procedure are presented as **Figures 6-7 and 6-8**, respectively.

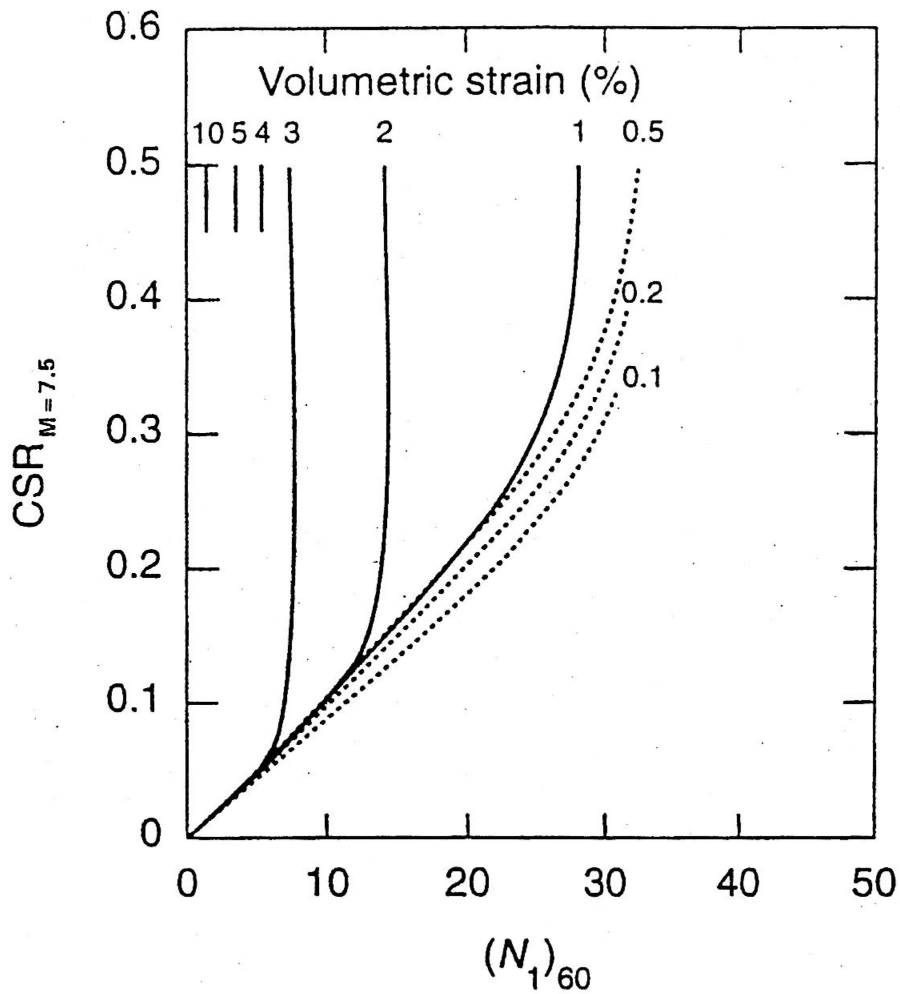


Figure 6-7 Liquefaction induced settlement estimated using the Tokimatsu & Seed procedure (Tokimatsu and Seed, 1987).

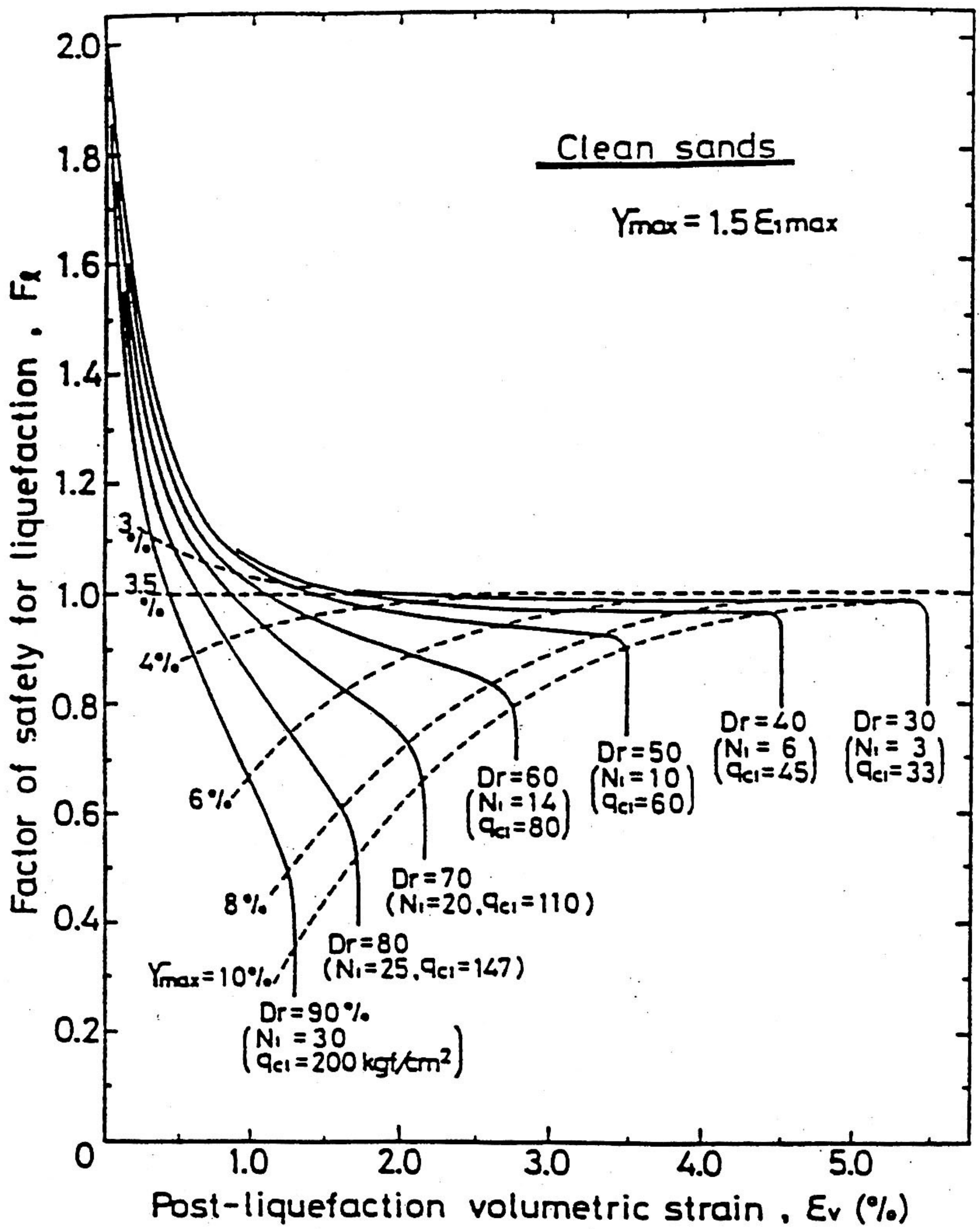


Figure 6-8 Liquefaction induced settlement estimated using the Ishihara and Yoshimine procedure (Ishihara and Yoshimine, 1992).

6.5.2.4 Residual Strength Parameters

Liquefaction induced instability is strongly influenced by the residual strength of the liquefied soil. Instability occurs when the shear stresses required to maintain equilibrium exceed the residual strength of the soil deposit. Evaluation of residual strength of a liquefied soil deposit is one of the most difficult problems in geotechnical practice (**Kramer, 1996**). A variety of methods are available to estimate the residual strength of liquefied soils; however, arguably the most widely accepted procedure, and the procedure recommended herein, is that proposed by **Seed and Harder, (1990)** which is presented in **Figure 6-4**.

The Seed and Harder procedure for estimating the residual strength of a liquefied soil deposit is based on an empirical relationship between residual undrained shear strength and equivalent clean sand SPT blowcounts. This relationship is based on back-calculation of the apparent shear strengths from case histories of flow slides. The Seed and Harder approach yields a range of residual undrained shear strength values for a given corrected SPT N value. Residual undrained shear strength values from the lower portion of the estimated range should be used for design.

6.5.2.5 Flow Failures and Lateral Spreading

Liquefaction Induced Flow Failure: Liquefaction can lead to catastrophic flow failures. Flow failures are driven by large static stresses that lead to large deformation or flow following triggering of liquefaction. Such failures are similar to debris flows. Flow failures are characterized by sudden initiation, rapid failure, and the large distances over which the failed materials move (**Kramer, 1996**). Flow failures typically occur during or shortly after shaking. However, delayed flow failures caused by post-earthquake redistribution of pore water pressures can occur—particularly if liquefiable soils are capped by relatively impermeable layers. For flow failures, both stability and deformation should be assessed and mitigated if stability failure or excessive deformation is predicted.

The potential for liquefaction induced flow failures is most often evaluated using conventional limit equilibrium slope stability analyses using residual undrained shear strength parameters for the liquefied soil and modeling the slope failure as an infinite slope or as a block failure. Where the factor of safety is less than unity, flow failure shall be considered likely. In these instances, the magnitude of deformation is usually too large to be acceptable for design of bridges or structures, and some form of mitigation is appropriate. The exception is where the liquefied material and crust flow past the structure and the structure can accommodate the imposed loads (see **WSDOT GDM Section 6.4.4**). Where the factor of safety is greater than unity for static conditions, deformations can be estimated using a Newmark type analysis or the **Youd et al. (2002)** empirical approach.

Lateral Spreading. In contrast to flow failures, lateral spreading results when the shear strength of the liquefied soil is incrementally exceeded by the inertial forces induced during an earthquake. The result of lateral spreading is typically horizontal movement of non-liquefied soils located above liquefied soils, in addition to the liquefied soils themselves.

The potential for liquefaction induced lateral spreading on gently sloping sites or where the site is located near a free face should be evaluated using empirical relationships such as the procedure of **Youd et al. (2002)**. This procedure (also known as the Bartlett and Youd procedure) uses empirical relationships based on case histories of lateral spreading. Input into the Youd et al. model includes earthquake magnitude, source-to-site distance, site geometry/slope, cumulative thickness of saturated soil layers with corrected SPT N values less than 15, average fines content and average grain size of these layers. The Youd et al. procedure provides a useful index of the potential magnitude of deformation; however, in many instances, further analyses through limit equilibrium or other means may be necessary for design. **Youd et. al. (2002)** present equations for estimating lateral spreading at sites with a free face condition as well as those with sloping ground.

6.5.3 Slope Instability

Slope instability can be due to inertial effects associated with ground accelerations, liquefaction or increased pore water pressures in slopes associated with a design seismic event, or both. Slope instability can also be initiated during a seismic event due to the weakening of sensitive fine grains soils. If liquefiable soils are present below embankments or within cut slopes, rapid strength loss in the liquefied soils could result in the initiation of a general slope failure. The liquefiable layer(s) shall be assigned residual strength parameters consistent with **WSDOT GDM Section 6.5.2.4**. When using liquefied soil shear strengths, the horizontal and vertical pseudo-static coefficients, k_h and k_v, respectively, should be set equal to zero, unless the earthquake controlling the seismic design is a very long duration earthquake, such as from the CSZ Interplate Source Zone as described in **WSDOT GDM Section 6-A.1.2**. For very long duration earthquakes, k_h should be set to 0.33PGA for this stability analysis. For these conditions resulting from a seismic event, the target factor of safety or resistance factor are as specified in **WSDOT GDM Section 6.5.3.1**.

6.5.3.1 Pseudo-static Analysis

Pseudo-static slope stability analyses should be used to evaluate the seismic stability of slopes and embankments. The pseudo-static analysis consists of conventional limit equilibrium static slope stability analysis as described in **WSDOT GDM Chapter 7** completed with horizontal and vertical pseudo-static acceleration coefficients (k_h and k_v) that act upon the critical failure mass. **Kramer (1996)** provides a detailed summary on pseudo-static analysis.

A horizontal pseudo-static coefficient, k_h, of 0.5PGA and a vertical pseudo-static coefficient, k_v, equal to zero should be used when seismic stability of slopes is evaluated not considering liquefaction. For these conditions, the target factor of safety is 1.1. When bridge foundations or retaining walls are involved, the LRFD approach shall be used, in which case a resistance factor of 0.9 would be used for slope stability, and the slope would be designed at the service limit state (see **WSDOT GDM Chapters 8 and 15**).

6.5.3.2 Deformations

Deformation analyses should be employed where an estimate of the magnitude of seismically induced slope deformation is required. Acceptable methods of estimating the magnitude of seismically induced slope deformation include Newmark sliding block analysis, simplified charts based on Newmark-type analyses (**Makdisi and Seed, 1978 or Bray and Rathje, 1998**), or dynamic stress-deformation models. These methods should not be employed to estimate displacements associated with liquefaction or cyclic strength loss if the static factor of safety with the reduced strength parameters is less than unity.

Newmark Analysis. Newmark (1965) proposed a seismic slope stability analysis that provides an estimate of seismically induced slope deformation. The advantage of the Newmark analysis over pseudo-static analysis is that it provides an index of permanent deformation. The Newmark analysis treats the unstable soil mass as a rigid block on an inclined plane. The procedure for the Newmark analysis consists of three steps that can generally be described as follows:

- Identify the yield acceleration of the slope by completing limit equilibrium stability analyses. The yield acceleration is the horizontal pseudo-static coefficient, k_h, required to bring the factor of safety to unity.
- Select an earthquake time history representative of the design earthquake.
- Double integrate all relative accelerations (i.e., the difference between acceleration and yield acceleration) in the earthquake time history.

A number of commercially available computer programs are available to complete Newmark analysis, such as Shake 2000 (**Ordoñez, 2000**) or Java Program for using Newmark Method and Simplified Decoupled Analysis to Model Slope Deformation During Earthquakes (**Jibson, 2003**).

Makdisi-Seed Analysis. Makdisi and Seed (1978) developed a simplified procedure for estimating seismically induced slope deformations based on Newmark sliding block analysis. The Makdisi-Seed procedure provides an estimated range of permanent seismically induced slope deformation as a function of the ratio of yield acceleration over maximum acceleration and earthquake magnitude as shown on **Figure 6-9**. The Makdisi-Seed procedure provides a useful index of the magnitude of slope deformation. Because the Makdisi-Seed procedure includes the dynamic effects of the seismic response of dams, its results should be interpreted with caution when applied to other slopes.

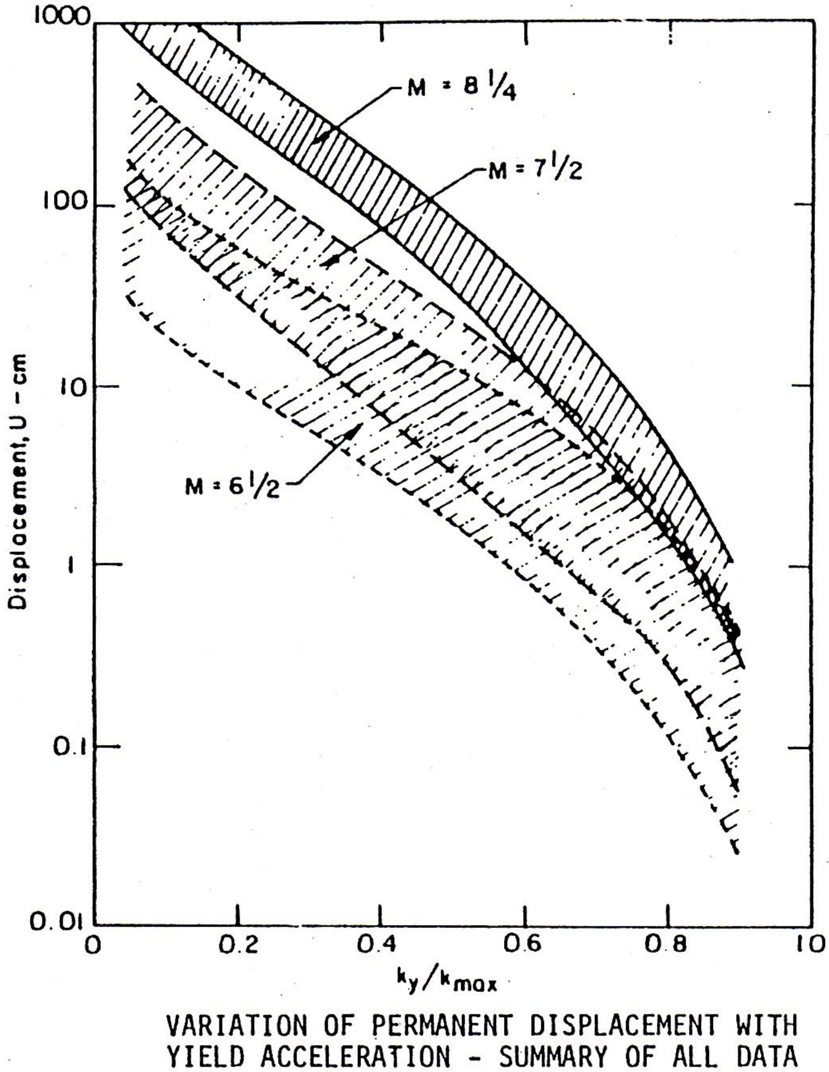


Figure 6-9 The Makdisi-Seed procedure for estimating the range of permanent seismically induced slope deformation as a function of the ratio of yield acceleration over maximum acceleration (Makdisi and Seed, 1978).

Bray-Rathje Analysis. **Bray and Rathje (1998)** developed an approach to estimate permanent base sliding deformation for solid waste landfills. The method is based on the Newmark sliding block model, and is similar to the Makdisi-Seed approach. However, the Bray-Rathje charts are based on significantly more analyses and a wider range of earthquake magnitudes, peak ground accelerations and frequency content than the Makdisi-Seed charts and may be more reliable. A Bray-Rathje chart depicting permanent base deformation as a function of yield acceleration (K_y) over the maximum horizontal equivalent acceleration (K_{max}) acting on the slide mass is presented in **Figure 6-10**. See **Bray and Rathje (1998)** for additional discussion regarding the determination of K_{max}.

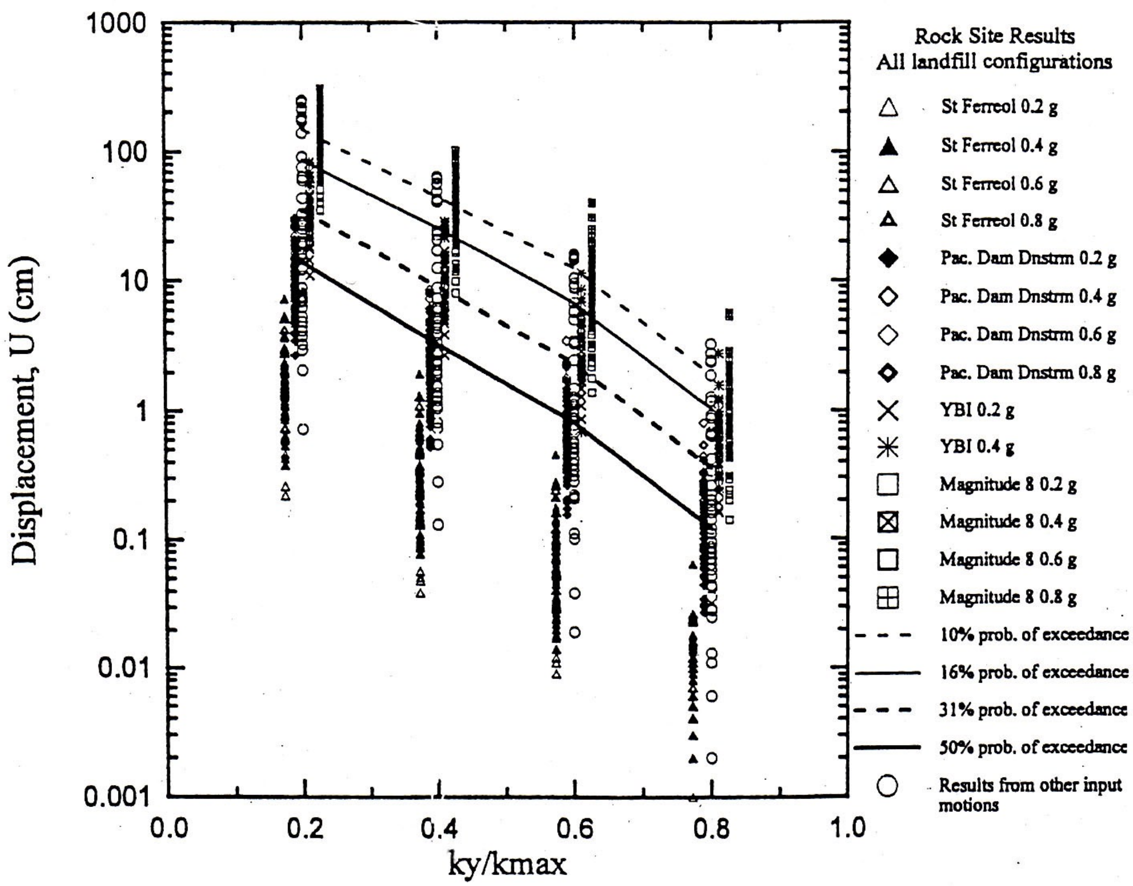


Figure 6-10 Permanent Base Sliding Block Displacements as a Function of Yield Acceleration to Maximum Horizontal Equivalent Acceleration (Bray and Rathje, 1998).

Dynamic Stress-Deformation Models. Seismically induced slope deformations can be estimated through a variety of dynamic stress-deformation computer models such as PLAXIS, DYNAFLOW, and FLAC. The accuracy of these models is highly dependent upon the quality of the input parameters. As the quality of the constitutive models used in dynamic stress-deformation models improves, the accuracy of these methods will improve. Another benefit of these models is their ability to illustrate mechanisms of deformation, which can provide useful insight into the proper input for simplified analyses.

Dynamic stress deformation models should not be used for routine design due to their complexity, and due to the sensitivity of the accuracy of deformation estimates from these models on the constitutive model selected and the accuracy of the input parameters. Use of dynamic stress-deformation models for design on WSDOT projects shall be approved by the WSDOT State Geotechnical Engineer.

6.5.4 Settlement of Dry Sand

Seismically induced settlement of unsaturated granular soils (dry sands) is well documented. Factors that affect the magnitude of settlement include the density and thickness of the soil deposit and the magnitude of seismic loading. The most common means of estimating the magnitude of dry sand settlement are through empirical relationships based on procedures similar to the Simplified Procedure for evaluating liquefaction susceptibility. The procedures provided by **Tokimatsu and Seed (1987)** for dry sand settlement should be used. The Tokimatsu and Seed approach estimates the volumetric strain as a function of cyclic shear strain and relative density or normalized SPT N values. The step by step procedure is presented in Section 8.5 of Geotechnical Engineering Circular No. 3 (**Kavazanjian, et al., 1997**).

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Appendix 6-A Site Specific Seismic Hazard and Site Response

Site specific analysis shall be completed where required by specification or where geologic conditions may result in un-conservative results if the generalized code hazard and response spectra are used. Special studies to determine site acceleration coefficients may be required where the site is located close to a fault, long-duration ground motion is expected, or if the importance of the bridge is such that a longer exposure period is required. When site specific hazard characterization is conducted, it shall be conducted using the design risk levels specified in **WSDOT GDM Section 6.3.1**.

6-A.1 Background Information for Performing Site Specific Analysis

Washington State is located in a seismically active region. The seismicity varies throughout the state, with the seismic hazard generally more severe in Western Washington and less severe in Eastern Washington. Earthquakes as large as magnitude 8 to 9 are considered possible in Washington State. The regional tectonic and geologic conditions in Washington State combine to create a unique seismic setting. Washington State has many faults that can produce damaging earthquakes; however, many of these faults have not been identified or have not generated earthquakes in recent geologic time. Where historic or geologic data is available, the seismic hazard from a specific fault can be quantified. Where limited historic or geologic data is available, the seismicity is characterized more in terms of earthquake source zones rather than by individual faults. A clear understanding of the regional tectonic setting and the recognized seismic source zones is essential for characterizing the seismic hazard at a specific site in Washington State.

6-A.1.1 Regional Tectonics

Washington State is located at the convergent continental boundary known as the Cascadia Subduction Zone (CSZ). The CSZ is the zone where the westward advancing North American Plate is overriding the subducting Juan de Fuca Plate. The CSZ extends from mid-Vancouver Island to Northern California. The interaction of these two plates results in three potential seismic source zones as depicted on **Figure 6-A-1**. These three seismic source zones are: (1) the shallow crustal source zone, (2) the Benioff source zone, and (3) the CSZ interplate source zone.

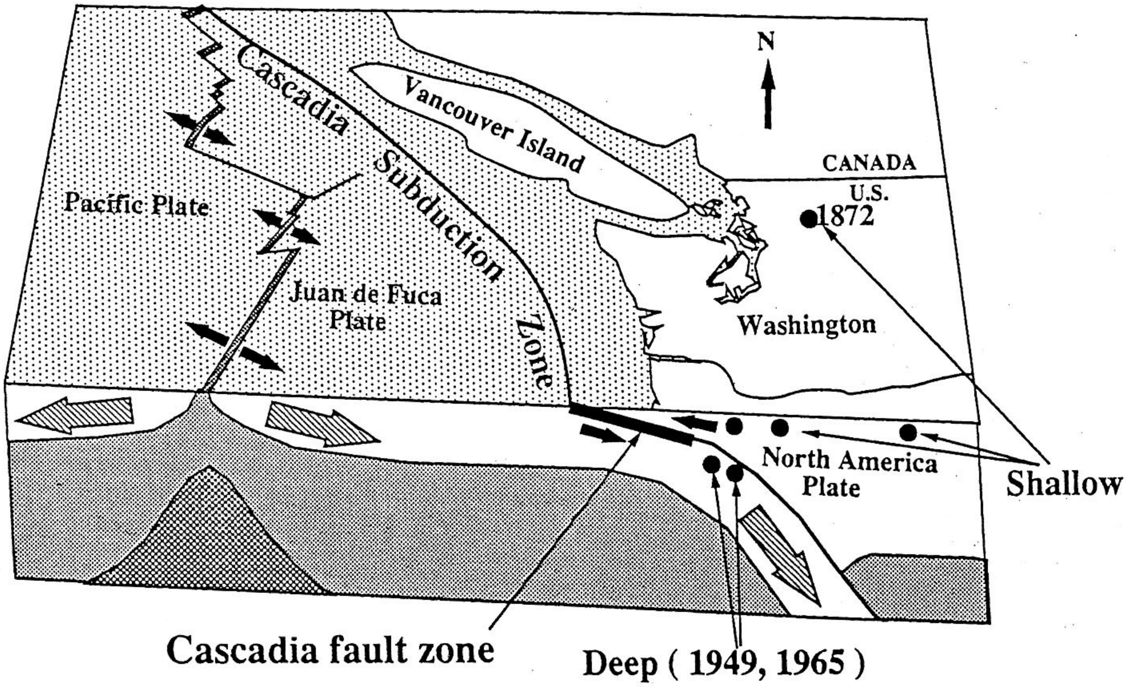

Figure 6-A-1 The three potential seismic source zones present in the Pacific Northwest (Yelin et al., 1994).

6-A.1.2 Seismic Source Zones

If conducting a site specific hazard characterization, as a minimum, the following source zones should be evaluated (all reported magnitudes are moment magnitudes):

Shallow Crustal Source Zone. The shallow crustal source zone is used to characterize shallow crustal earthquake activity within the North American Plate throughout Washington State. Shallow crustal earthquakes typically occur at depths ranging up to 12 miles. The shallow crustal source zone is characterized as being capable of generating earthquakes up to about magnitude 7.5. Large shallow crustal earthquakes are typically followed by a sequence of aftershocks.

The largest known earthquakes associated with the shallow crustal source zone in Washington State include an event on the Seattle Fault about 900 AD and the 1872 North Cascades earthquake. The Seattle Fault event was believed to have been magnitude 7 or greater (**Johnson, 1999**), and the 1872 North Cascades earthquake is estimated to have been between magnitudes 6.8 and 7.4. The location of the 1872 North Cascades earthquake is uncertain; however, recent research suggests the earthquake's intensity center was near the south end of Lake Chelan (**Bakun et al, 2002**). Other large, notable shallow earthquakes in and around the state include the 1936 Milton Freewater, Oregon magnitude 6.1 earthquake and the North Idaho magnitude 5.5 earthquake (**Goter, 1994**)

Benioff Source Zone. Benioff source zone earthquakes are also referred to as intraplate, intraslab, or deep subcrustal earthquakes. Benioff zone earthquakes occur within the subducting Juan de Fuca Plate between depths of 20 and 40 miles and typically have no large aftershocks. Extensive faulting results as the Juan de Fuca Plate is forced below the North American plate and into the upper mantle. Benioff zone earthquakes primarily contribute to the seismic hazard within Western Washington.

The Olympia 1949 (M = 7.1), the Seattle 1965 (M = 6.5), and the Nisqually 2001 (M = 6.8) earthquakes are considered to be Benioff zone earthquakes. The Benioff zone is characterized as being capable of generating earthquakes up to magnitude 7.5. The recurrence interval for large earthquakes originating from the Benioff source zone is believed to be shorter than for the shallow crustal and CSZ source zones—damaging Benioff zone earthquakes in Western Washington occur every 30 years or so. The deep focal depth of these earthquakes tends to dampen the shaking intensity when compared to shallow crustal earthquakes of similar magnitudes.

CSZ Interplate Source Zone. The Cascadia Subduction Zone (CSZ) is an approximately 650-mile long thrust fault that extends along the Pacific Coast from mid-Vancouver Island to Northern California. CSZ interplate earthquakes result from rupture of all or a portion of the convergent boundary between the subducting Juan de Fuca plate and the overriding North American plate. The fault surfaces approximately 50 to 75 miles off the Washington coast. The width of the seismogenic portion of the CSZ interplate fault varies along its length. As the fault becomes deeper, materials being faulted become ductile and the fault is unable to store mechanical stresses. CSZ earthquakes primarily contribute to the seismic hazard within Western Washington.

The CSZ is considered as being capable of generating earthquakes of magnitude 8 to magnitude 9. No earthquakes on the CSZ have been instrumentally recorded; however, through the geologic record and historical records of tsunamis in Japan, it is believed that the most recent CSZ event occurred in the year 1700 (**Atwater, Brian F, 1996 and Satake, K, et al, 1996**). Recurrence intervals for CSZ interplate earthquakes are thought to be on the order of 400 to 600 years. Paleogeologic evidence suggests five to seven interplate earthquakes may have been generated along the CSZ over the last 3,500 years at irregular intervals.

6-A.2 Design Earthquake Magnitude

In addition to identifying the site's source zones, the design earthquake(s) produced by the source zones must be characterized for use in evaluating seismic geologic hazards such as liquefaction and lateral spreading. Typically, design earthquake(s) are defined by a specific magnitude, source-to-site distance, and PGA.

The following guidelines should be used for determining a site's design earthquake(s):

- The design earthquake should consider risk-compatible events occurring on crustal and subduction-related sources.
- More than one design earthquake may be appropriate depending upon the source zones that contribute to the site's seismic hazard and the impact that these earthquakes may have on site response.
- The design earthquake should be consistent with the design risk level prescribed in **WSDOT GDM Section 6.3.1**.

The USGS interactive deaggregation tool provides a summary of contribution to seismic hazard for earthquakes of various magnitudes and source to site distances for a given risk level and may be used to evaluate relative contribution to ground motion from seismic sources. The geotechnical designer may utilize this tool to aid in selecting input parameters for subsequent analysis such as ground motion modeling, time history development, liquefaction susceptibility, and lateral displacement analysis. Note that magnitudes presented in the deaggregation data represent contribution to a specified level of risk and should not be averaged for input into analyses such as liquefaction and lateral spreading. Instead, the deaggregation data should be used to assess the relative contribution to the probabilistic hazard from the various source zones. If any source zone contributes more than about 10 percent of the total hazard, design earthquakes representative from each of those source zones should be used for analyses. When a range of earthquake magnitudes and site to source distances are presented by the deaggregation data, the highest magnitude for the source zone should be selected. Default magnitudes (previously presented) for the source (or sources) representing greater hazard than shown in the deaggregation data could also be used in analyses.

6-A.3 Probabilistic and Deterministic Seismic Hazard Analyses

Probabilistic seismic hazard analysis (PSHA) and deterministic seismic hazard analysis (DSHA) can be completed to characterize the seismic hazard at a site. A DSHA consists of evaluating the seismic hazard at a site for an earthquake of a specific magnitude occurring at a specific location. A PSHA consists of completing numerous deterministic seismic hazard analyses for all feasible combinations of earthquake magnitude and source to site distance for each earthquake source zone. The result of a PSHA is a relationship of the mean annual rate of exceedance of the ground motion parameter of interest with each potential seismic source considered. Since the PSHA provides information on the aggregate risk from each potential source zone, it is more useful in characterizing the seismic hazard at a site if numerous potential sources could impact the site. The USGS 2002 probabilistic hazard maps on the USGS website are based on PSHA.

PSHAs and DSHAs may be required where the site is located close to a fault, long-duration ground motion is expected, or if the importance of the bridge is such that a longer exposure period is required by WSDOT. For a more detailed description and guidelines for development of PSHAs and DSHAs, see **Kramer (1996) and McGuire (2004)**.

Site specific hazard analysis should include consideration of topographic and basin effects, fault directivity and near field effects.

At a minimum, seismic hazard analysis should consider the following sources:

- Cascadia subduction zone interface earthquake
- Cascadia subduction zone intraplate earthquake
- Crustal earthquakes associated with non-specific or diffuse sources (potential sources follow). These sources will account for differing tectonic and seismic provinces and include seismic zones associated with Cascade volcanism
- Earthquakes on known and potentially active crustal faults. The following list of potential seismic sources may be used for hazard assessment and site response development. The applicability of these sources will depend on their proximity to the site.
    - Seattle Fault Zone
    - Southern Whidbey Island Fault
    - Utsalady Fault
    - Strawberry Point Fault
    - Devils Mountain Fault
    - Horse Heaven Hills Anticline
    - Rattlesnake-Wallula Fault System
    - Mill Creek Fault
    - Saddle Mountains Fault
    - Hite Fault System

When PSHA or DSHA are performed for a site, the following information shall be included in project documentation and reports:

- Overview of seismic sources considered in analysis
- Summary of seismic source parameters including length/boundaries, source type, slip rate, segmentation, maximum magnitude, recurrence models and relationships used, source depth and geometry. This summary will include the rationale behind selection of source parameters.
- Assumptions underlying the analysis will be summarized in either a table (DSHA) or in a logic tree (PSHA)

The 2002 USGS probabilistic hazard maps on the USGS website essentially account for regional seismicity and attenuation relationships, recurrence rates, maximum magnitude of events on know faults or source zones, and the location of the site with respect to the faults or source zones. The USGS data is sufficient for most sites, and more sophisticated seismic hazard analyses are generally not required; the exceptions may be to capture the effects of sources not included in the USGS model, to assess near field or directivity influences, or to incorporate topographic impacts.

6-A.4 Selection of Attenuation Relationships

Attenuation relationships describe the decay of earthquake energy as it travels from the seismic source to the project site. Many of the published relationships are capable of accommodating site soil conditions as well as varying source parameters (e.g., fault type, location relative to the fault, near-field effects, etc.) In addition, during the past 8 years, specific attenuation relationships have been developed for Cascadia subduction zone sources. For both deterministic and probabilistic risk assessments, attenuation relationships used in analysis should be selected based on applicability to both the site conditions and the type of seismic source under consideration. Rationale for the selection of and assumptions underlying the use of attenuation relationships for risk characterization shall be clearly documented.

6-A.5 Site Specific Response Analysis

6-A.5.1 Design/Computer Models

Site response analysis is generally based on the assumption of a vertically propagating shear wave through uniform soils. The influence of vertical motions, compression waves, laterally non-uniform soil conditions, incoherence and spatial variation of ground motions are not accounted for in conventional site response analyses (**Kavazanjian, et al., 1997**). A variety of site response computer models are available to geotechnical designers for dynamic site response analyses. In general, there are three general classes of site response models: 1) equivalent linear, 2) nonlinear, and 3) multi-dimension models.

Equivalent Linear Models. One-dimensional equivalent linear site response computer codes, such as ProShake (**EduPro Civil Systems, 1999**) or Shake2000 (**Ordoñez, 2000**), use an iterative total stress approach to estimate the nonlinear, inelastic behavior of soils. These programs use an average shear modulus over the entire cycle of loading to approximate the hysteresis loop.

The equivalent linear model provides reasonable results for small strains (less than about 1 to 2 percent) and modest accelerations (less than about 0.3 to 0.4g) (**Kramer and Paulsen, 2004**). Equivalent linear analysis cannot be used where large strain incompatibilities are present, to estimate permanent displacements, or to model development of pore water pressures.

Nonlinear Models. Nonlinear computer codes, such as PLAXIS, FLAC, DYNAFLOW, DMOD, or DESRA, use direct numerical integration of the equation of motion in small time steps and account for the nonlinear soil behavior through use of constitutive soil models. Depending upon the constitutive model used, these programs can model pore water pressure buildup and permanent deformations. The accuracy of nonlinear models depends on the quality of the parameters used by constitutive soil model and the reliability of the constitutive model.

Two and Three Dimensional Models. Two- and three-dimensional computer codes are available for both equivalent linear and nonlinear site response analysis. The attributes of the two- and three-dimensional models are similar to those described above for the one-dimensional equivalent linear and nonlinear models. However, the two- and three-dimensional computer codes require significantly more computational time than one-dimensional analyses. The primary advantage of the two- and three-dimensional models is that soil anisotropy, irregular soil stratigraphy, and irregular topography can be modeled. Another advantage with the two- and three-dimensional models is that seismically induced permanent displacements can be estimated.

6-A.5.2 Input Parameters for Site Specific Response Analysis

The input parameters required for site specific seismic response analysis include dynamic soil properties for each soil layer, the depth to bedrock or hypothetical equivalent bedrock layer, and ground motion time histories. Soil parameters required by the equivalent linear models include the shear wave velocity or initial (small strain) shear modulus and unit weight for each soil layer, and curves relating the shear modulus and damping ratio as a function of shear strain (See **Figures 6-1 through 6-3**).

Soil parameters required for the nonlinear models include the soil profile definition and parameters for the constitutive soil model. The parameters required for the constitutive soil model generally consist of a backbone curve that models the stress strain path during cyclic loading and rules for loading and unloading, stiffness degradation, and other factors (**Kramer, 1996**).

A suite of ground motion time histories are required for both equivalent linear and nonlinear site response analyses. The use of at least three input ground motions is recommended for site response analysis. Ground motion time histories can either be processed acceleration records from actual earthquake events or can be synthetically generated acceleration records; the use of actual earthquake records is preferred. Ideally, the parameters of the acceleration time histories used for analyses should correspond closely to the site conditions.

Acceleration time histories recorded during earthquakes on faults with similar fault mechanisms to those of recognized seismic source zones that contribute to the site's seismic hazard should be selected for site specific response analysis. Also, if the earthquake records are used in the site response model as bedrock motion, the records should be recorded on sites with bedrock characteristics. The frequency content, earthquake magnitude, and peak bedrock acceleration should also be used as criteria to select earthquake time histories for use in site response analysis. Input ground motions should be baseline corrected and scaled for use. Finally, for analyses where earthquake time histories at the ground surface will be computed for use by the structural engineer in nonlinear structural analysis, consideration should be given to using orthogonal pairs of natural earthquake acceleration time histories. Caution should be exercised when considering the use of synthetic records where earthquake time histories will be used in a two- or three-dimensional nonlinear structural analysis, as realistic orthogonal pairs are typically not available. All time histories to be used in structural analysis should be compared to or matched to the anticipated site response spectra.